CALIFORNIA INSTITUTE OF TECHNOLOGY

EARTHQUAKE ENGINEERING RESEARCH LABORATORY

DYNAMIC RESPONSES OF SIX MULTISTORY BUILDINGS DURING THE SAN FERNANDO EARTHQUAKE

By
Douglas A. Foutch
George W. Housner
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Report No. EERL 75-02

A Report on Research Conducted under Grants from the National Science Foundation and the Earthquake Research Affiliates Program at the California Institute of Technology

Pasadena, California

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Introduction

The development of optimal methods of earthquake resistant design depends on a knowledge of how the ground shakes and how real buildings oscillate during earthquakes. The San Fernando earthquake of February 9, 1971 occurred in a region well instrumented to record ground shaking and building vibration and, as a consequence, many records were obtained that illustrate the engineering features of earthquake ground motion and its effects. The magnitude 6.5 earthquake was centered on the northern edge of the Los Angeles Metropolitan area. Although the shock was not a great earthquake in seismological terms, it produced very strong ground shaking and, therefore, was an important event for engineers. The resulting damage, estimated at over $500 million, provided valuable engineering lessons concerning the design of buildings, bridges, dams, and other facilities; and numerous special studies of the effects of the San Fernando earthquake on various structures have been published.[1-6] The shock triggered 272 accelerographs, and 241 accelerograms were recorded.[1,7,9] These records provide valuable information about the characteristics of strong ground motions and structural responses, and make it possible to evaluate the measured performance of modern buildings during strong earthquakes.

The San Fernando earthquake was of special importance to structural engineers since it provided the first sample of recorded responses of modern multistoried buildings to strong earthquake motions. The responses of more than 50 buildings were measured, most of which were located in the city of Los Angeles, and had accelerographs in the basement, at mid-height, and on the roof in accordance with building code requirements. These recorded motions enabled studies to be made of modeling and analysis procedures, of the relationships between building response and damage, and of other important aspects of the response of tall structures to earthquake motions.[1-6]

It is very informative to structural engineers to examine the measured motions of buildings of different types and to compare them with the ground motions that caused the structural oscillations. The recorded accelerations, velocities, and displacements of all strong U.S. earthquakes are contained in Caltech reports[9] but these do not contain descriptions of the structures in which the motions were recorded. Therefore, in this report there are presented the responses and descriptions of selected structures of various types. These were chosen to be informative in a practical sense, that is, to demonstrate how representative buildings oscillate during earthquakes.

The main purpose of this report is to provide the practicing engineer with a collection of brief descriptions of a variety of multistory buildings and their responses to the San Fernando earthquake, and also to provide a list of references where more detailed information about these and other buildings can be found. Of the six buildings included in this report, three are steel frame buildings, two are reinforced concrete frame buildings and one is a reinforced concrete shear wall building. The buildings range from 7 to 42 stories in height and are located at distances of approximately 8 to 21 miles from the center of the San Fernando earthquake. More detailed studies of the earthquake responses of five of the buildings have been completed and the results have been published.[3,8] A study of the sixth building (Millikan Library) is presently underway. Table 1 is a list of the buildings along with some descriptive information about each such as size, location, construction, etc. Figure I.1 is a map of the Los Angeles area with the location of the buildings indicated.

Table 1

Building	No. of Stories	Type of Bldg.	Peak Absolute Motion* acceleration (% g) displacement (inches)	Approx. Dis. to Center of Energy Release of Earthquake (miles)	Type of Foundation
1. Union Bank Bldg. 445 Figueroa Street	42	Steel Frame	20% g 9.2 inches	21	Spread footings
2. Kajima International Bldg. 250 E. First	15	Steel Frame	19 8.8	21	Spread footings
3. Bank of California Bldg. 15250 Ventura Blvd.	12	Reinforced Concrete Frame	29 12.1	14	Cast-in-place concrete piles
4. Holiday Inn 8244 Orion Avenue	7	Reinforced Concrete Frame	38 9.5	8	Poured, reinforced concrete friction piles
5. Millikan Library Bldg. California Inst. of Tech.	9	Reinforced Concrete Shear Wall	35 4.6	19	Spread footings
6. Building 180 Jet Propulsion Laboratory	9	Steel Frame	38 3.7	15	Continuous strip footings

*Absolute motion is the sum of the motion of the ground plus the relative motion between the ground and the floor. Peak motions were recorded at the roof for all buildings except Union Bank where values at the 19th floor are given.

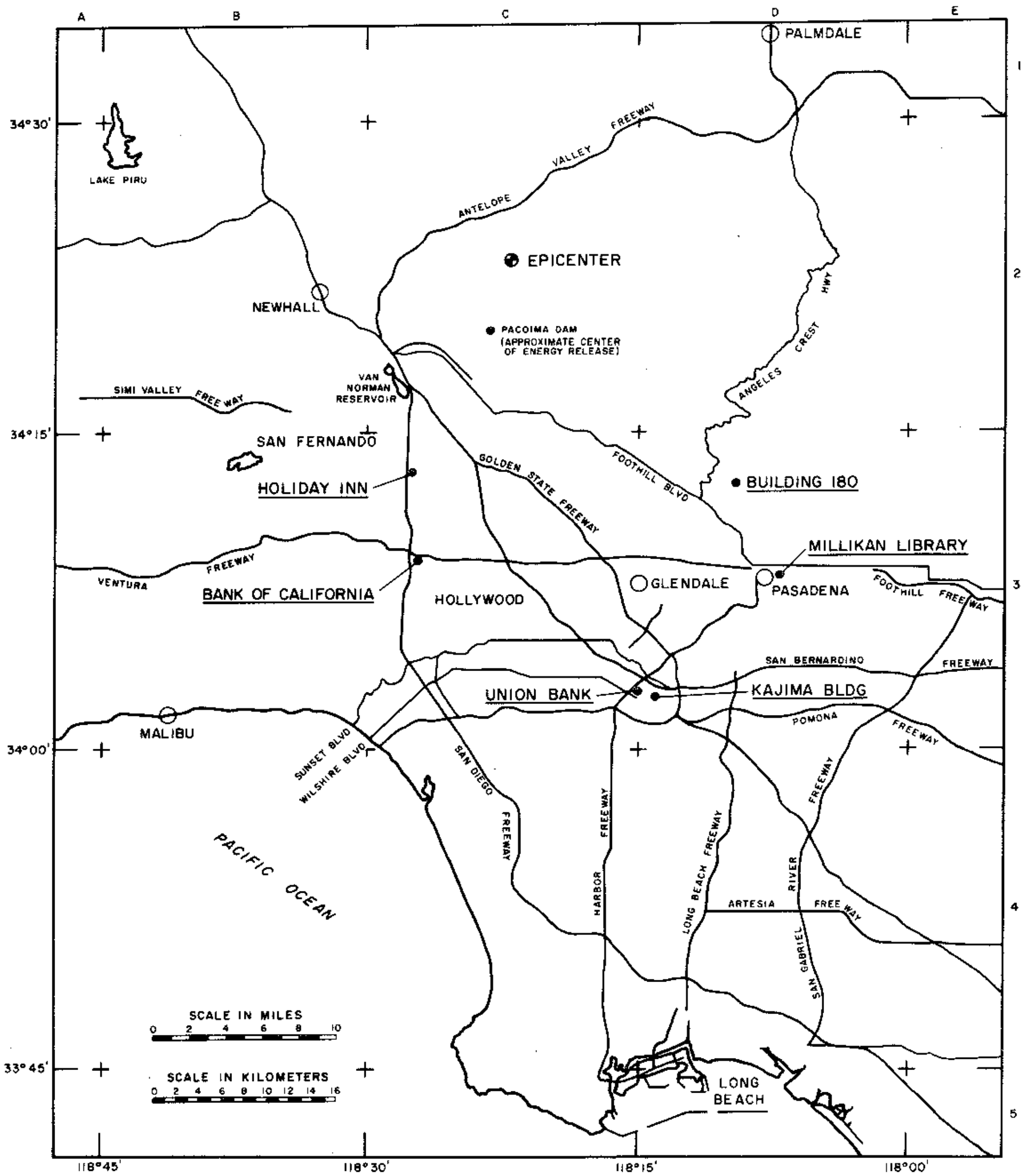

Figure I. 1 Map of Los Angeles area showing location of buildings

The individual report on each building begins with a two-to-three paragraph description of the building which includes its location, physical size, type of construction, damage experienced during the earthquake and lateral load resisting system. A paragraph is also included which describes the nature of the recorded ground motion at the site and the resulting building response. This is followed by a list of references of various studies that have been made of the building. In each case, the first reference listed has been used as a source of information for the building descriptions found in this report. These descriptions include a photograph of the building and a section and plan view of the structural system. The figures are followed by the accelerations recorded in the building during the earthquake along with the integrated displacement records. These motions represent the total movement of the building which is composed of the relative motion of the building with respect to the ground plus the motion of the ground. Also included are the two horizontal components of the relative displacement of the roof and mid-height motion where available. The relative displacements were determined by subtracting the displacement of the ground from the total displacement of the building.

The accelerographs that record the ground and building motions measure the absolute acceleration within an accuracy of a few percent, so that the recorded accelerograms may be considered very accurate for engineering purposes. When integrating the acceleration to obtain the velocity the accuracy is degraded; and it is further degraded when the velocity is integrated to obtain the displacement. The error in the displacement is most apparent in long period motion and because of this components of motion above 16 seconds period have been filtered out of the accelerograms.

The accelerograph remains at rest until the trigger is activated by the seismic waves, thus a small portion of the initial phase of ground shaking is lost. Because of this the data processing technique results in some distortion of the displacement curve in the initial few seconds of the displacement diagram, depending upon how much motion was lost.

References

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2. Jennings, P. C. (Ed.), Engineering Features of the San Fernando Earthquake, Earthquake Engineering Research Laboratory, Report No. 71-02, California Institute of Technology, Pasadena, California, 1971.

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8. Wood, J.H., Analysis of the Earthquake Response of a Nine Story Steel Frame Building During the San Fernando Earthquake, Earthquake Engineering Research Laboratory Report No. 72-04, California Institute of Technology, Pasadena, California, 1972.

9. "Strong Motion Earthquake Accelerograms", Vols. I, II, III, Earthquake Engineering Research Laboratory, California Institute of Technology, Pasadena, California.

Chapter 1

Union Bank Building
445 Figueroa Street
Los Angeles, California

Union Bank Square is located in downtown Los Angeles, 21 miles south of the center of the San Fernando earthquake of February 9, 1971. The 42-story Union Bank building tower, which was designed in 1964, is 98 by 196 feet in plan and stands 535 feet above the second basement level and 496 feet, 39 stories, above the adjacent plaza level. The top floor contains air handling equipment and the two basement levels are used for storage, office services and parking. The street level is rental space and the remaining floors are used as offices. Figure 1.1 is a photograph of the east side of the Union Bank building. The building sustained only minor nonstructural damage during the earthquake, such as plaster cracking around the elevators and tile damage in the restrooms.

The building had been designed on the basis of a dynamic analysis, and the lateral load resisting system of the tower is composed entirely of 100% moment resisting frames with welded connections. In the transverse direction wind and earthquake forces are resisted by six interior frames, four six-column and two five-column frames, plus two exterior frames, all of approximately equal stiffness. In the longitudinal direction two sidewall frames provide lateral resistance. Columns where transverse and longitudinal frames intersect were designed to participate in both directions with the exception of the four corner colums which were designed to participate only in the longitudinal direction. Figure 1.2a is a typical transverse section of the tower. Figure 1.2b is a typical floor plan with the heavy lines indicating the frames that participate in the lateral load resisting system. All other frames are non-moment resisting (not shown in figure). In addition to the frames, shear walls are provided from the second floor to the basement level to restrict drift and to increase the energy absorption capability of the system. The five-column transverse frames are supported by individual spread footings. The rest of the system is supported by continuous spread footings. A seismic separation is provided between the tower and adjoining plaza.

Strong-motion accelerographs were installed on the second basement, 19th floor and 39th floor. These were designed to start simultaneously when any one of the three reached the threshold level of motion; however, the instrument on the 39th floor failed to record. Three components of acceleration were recorded at the basement and 19th floor levels and these traces are presented in the following figures along with the integrated displacements and the computed relative displacements of the two horizontal components. Also presented are the response spectra of the ground motion recorded at the second basement level.

It can be seen in the seismograms that the Union Bank building vibrated with a period of approximately 4.5 seconds, which presumably is the natural period of the fundamental mode of vibration. The building was strongly excited by a large pulse in the ground motion at approximately 10 seconds. This pulse shows very clearly in the displacement record and less clearly in the acceleration record of the ground. Such a large pulse is not commonly seen in recorded earthquake ground motions. Presumably, it is the consequence of the particular faulting mechanism, the orientation of the fault with respect to the building site, and the characteristics of the geology between the fault and the site.

The period of vibration, 4.5 seconds, shown in the earthquake response is much longer than the period of 3.1 seconds found in the ambient vibration study.[2] The longer period is, however, near that calculated during the design of the structure. The loss of stiffness indicated by this period change is believed to be the result of cracking and other types of degradation of nonstructural elements during the higher level earthquake responses. The natural period after the earthquake was measured to be 4.1 seconds[3], which indicates that the loss of stiffness was not recovered.

The recorded accelerations on the 19th floor of the Union Bank building show that the higher modes of vibration were strongly excited during the first 10 seconds of the earthquake but following that the response was primarily in the fundamental mode. When the acceleration is integrated twice to give the displacement of the 19th floor, it is clearly seen that the floor displacements are primarily contributed by the fundamental mode vibration.

References

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2. Trifunac, M. D., Ambient Vibration Tests of a Thirty-Nine Story Steel Frame Building, Earthquake Engineering Research Laboratory, Report No. 70-06, Aug. 1970.

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Figure 1.1 Union Bank Square

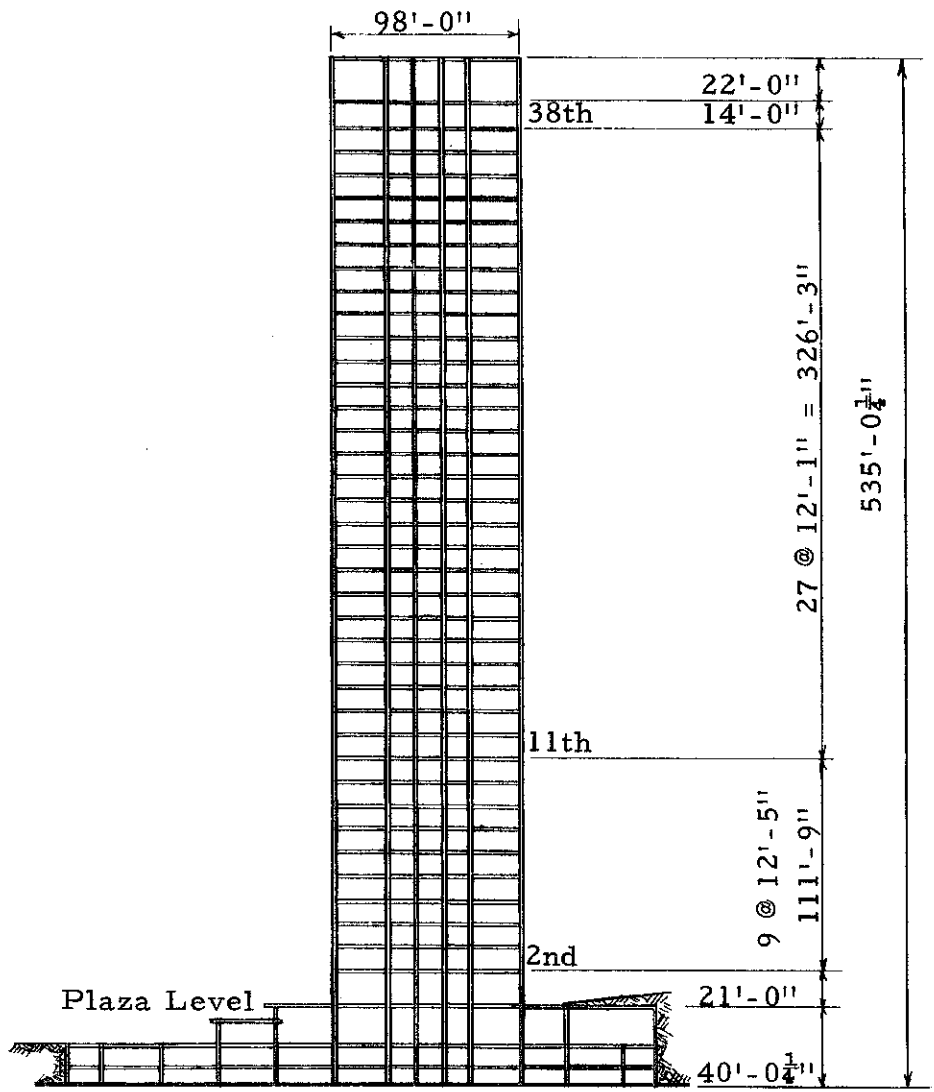


Figure 1.2a Transverse Section

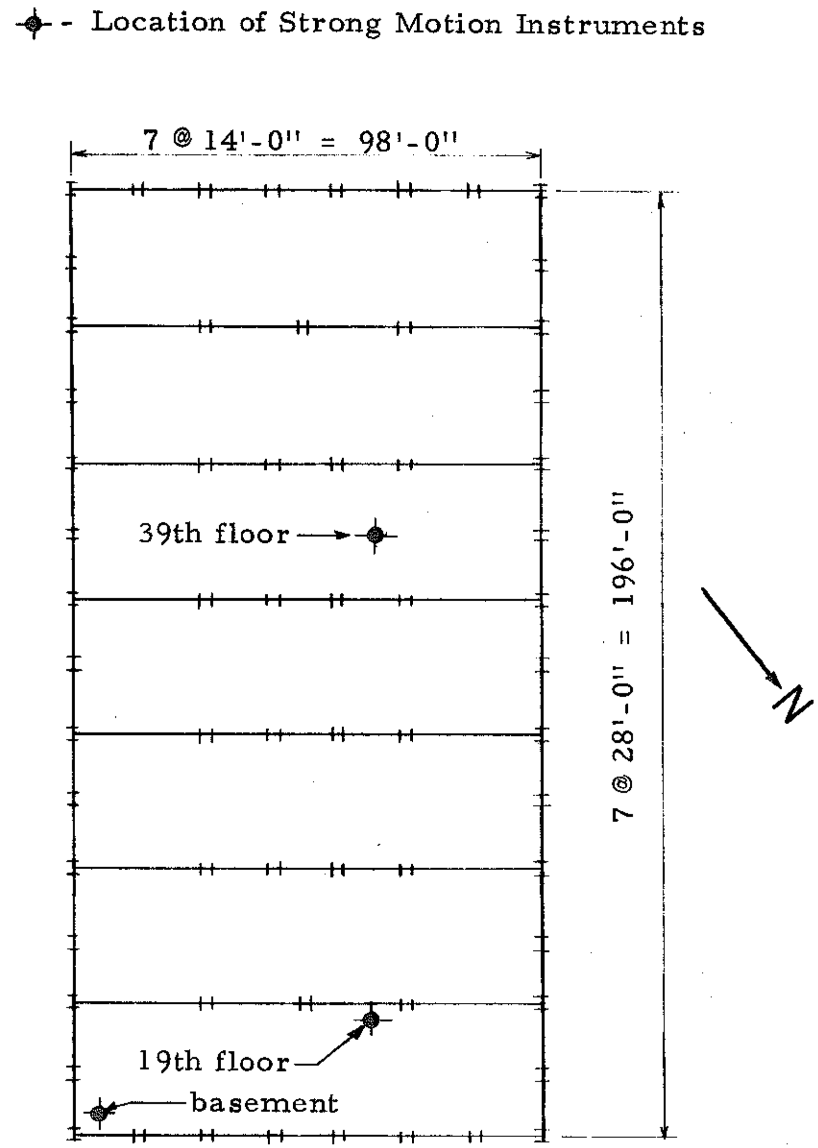


Figure 1.2b Typical Tower Floor Plan

Figure 1.2 Schematic of Union Bank Building Structural System

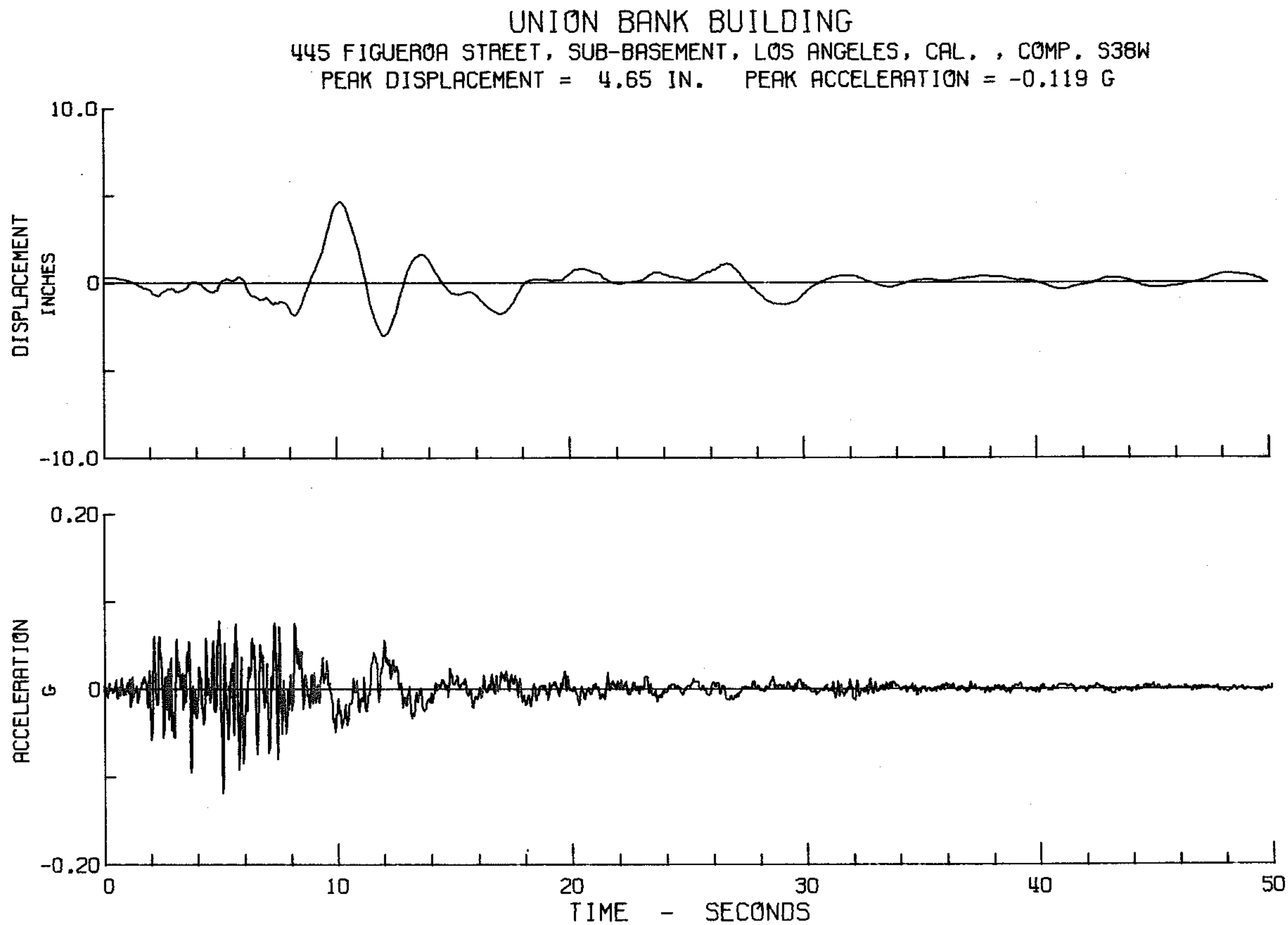

UNION BANK BUILDING
445 FIGUEROA STREET, SUB-BASEMENT, LOS ANGELES, CAL. , COMP. S38W
PEAK DISPLACEMENT = 4.65 IN. PEAK ACCELERATION = -0.119 G
DISPLACEMENT
INCHES
10.0
0
-10.0
ACCELERATION
G
0.20
0
-0.20
0
10
20
30
40
50
TIME - SECONDS

Figure 1.3

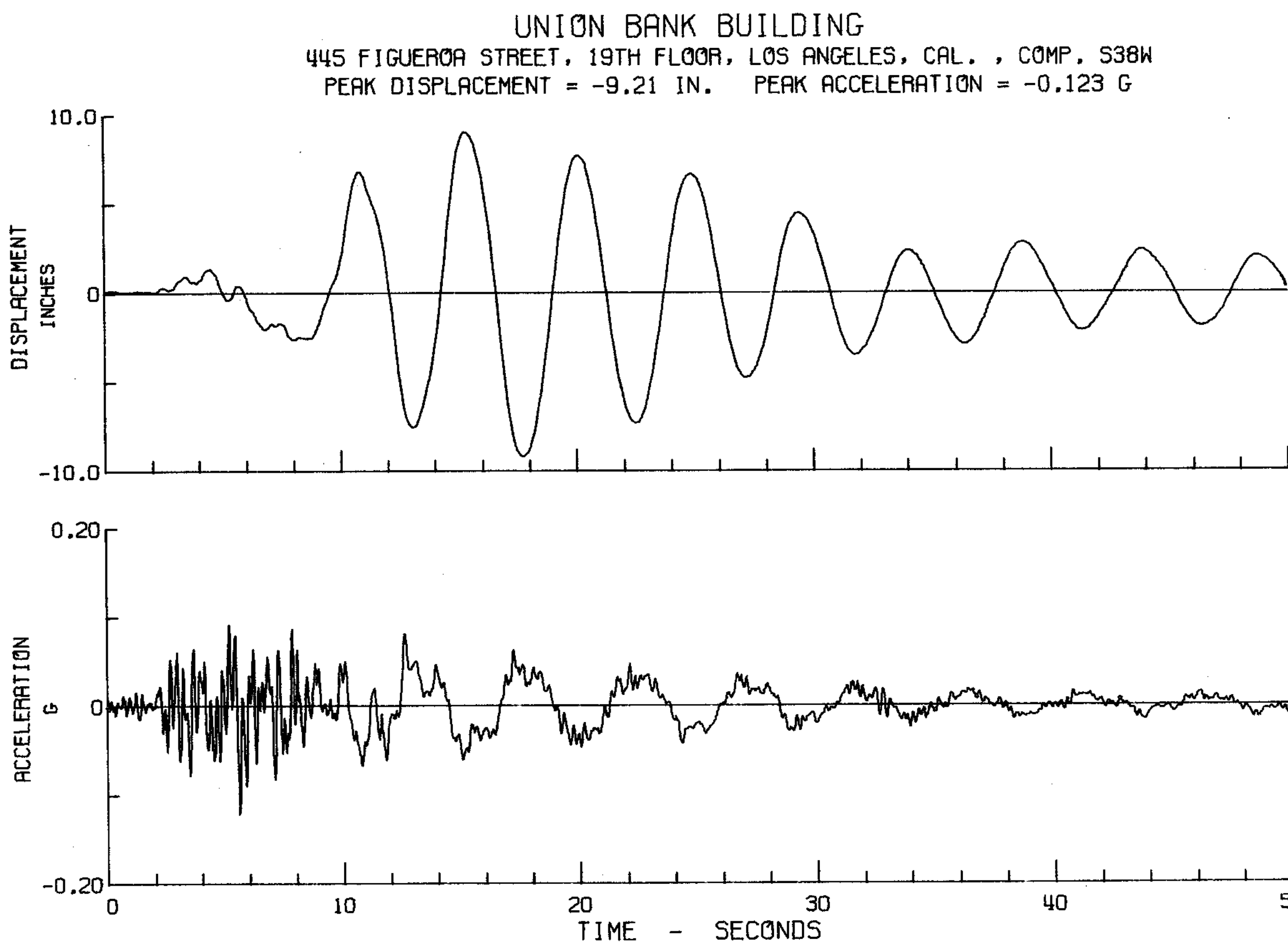

UNION BANK BUILDING
445 FIGUEROA STREET, 19TH FLOOR, LOS ANGELES, CAL. , COMP. S38W
PEAK DISPLACEMENT = -9.21 IN.   PEAK ACCELERATION = -0.123 G
DISPLACEMENT
INCHES
10.0
0
-10.0
ACCELERATION
G
0.20
0
-0.20
0
10
20
30
40
50
TIME - SECONDS

Figure 1.4

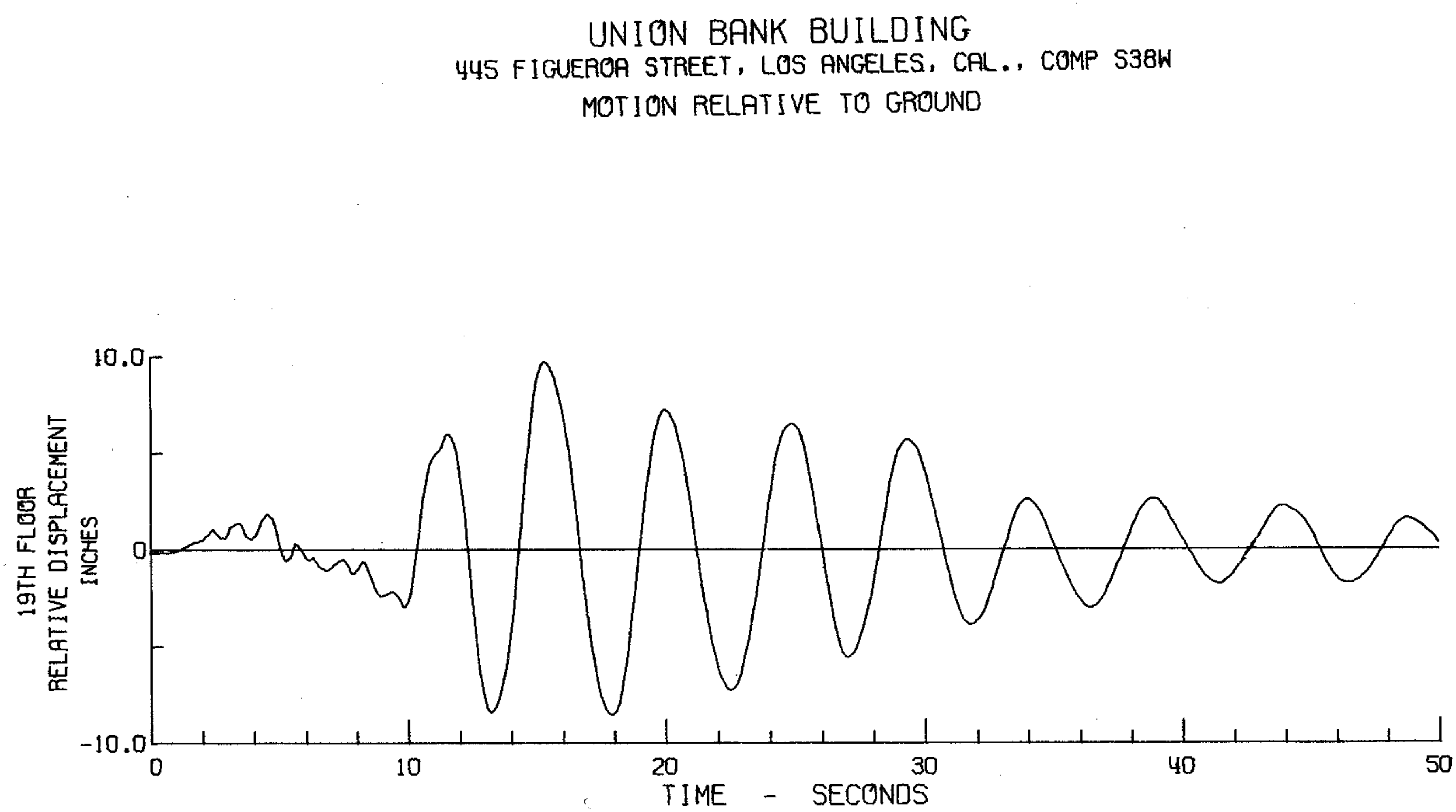

UNION BANK BUILDING
445 FIGUEROA STREET, LOS ANGELES, CAL., COMP S38W
MOTION RELATIVE TO GROUND
10.0
0
-10.0
19TH FLOOR
RELATIVE DISPLACEMENT
INCHES
0
10
20
30
40
50
TIME - SECONDS

Figure 1.5

RESPONSE SPECTRUM

UNION BANK BUILDING

445 FIGUEROA STREET, SUB-BASEMENT, LOS ANGELES, CAL., COMP. S38W

DAMPING VALUES ARE 0, 2, 5, 10 AND 20 PERCENT OF CRITICAL

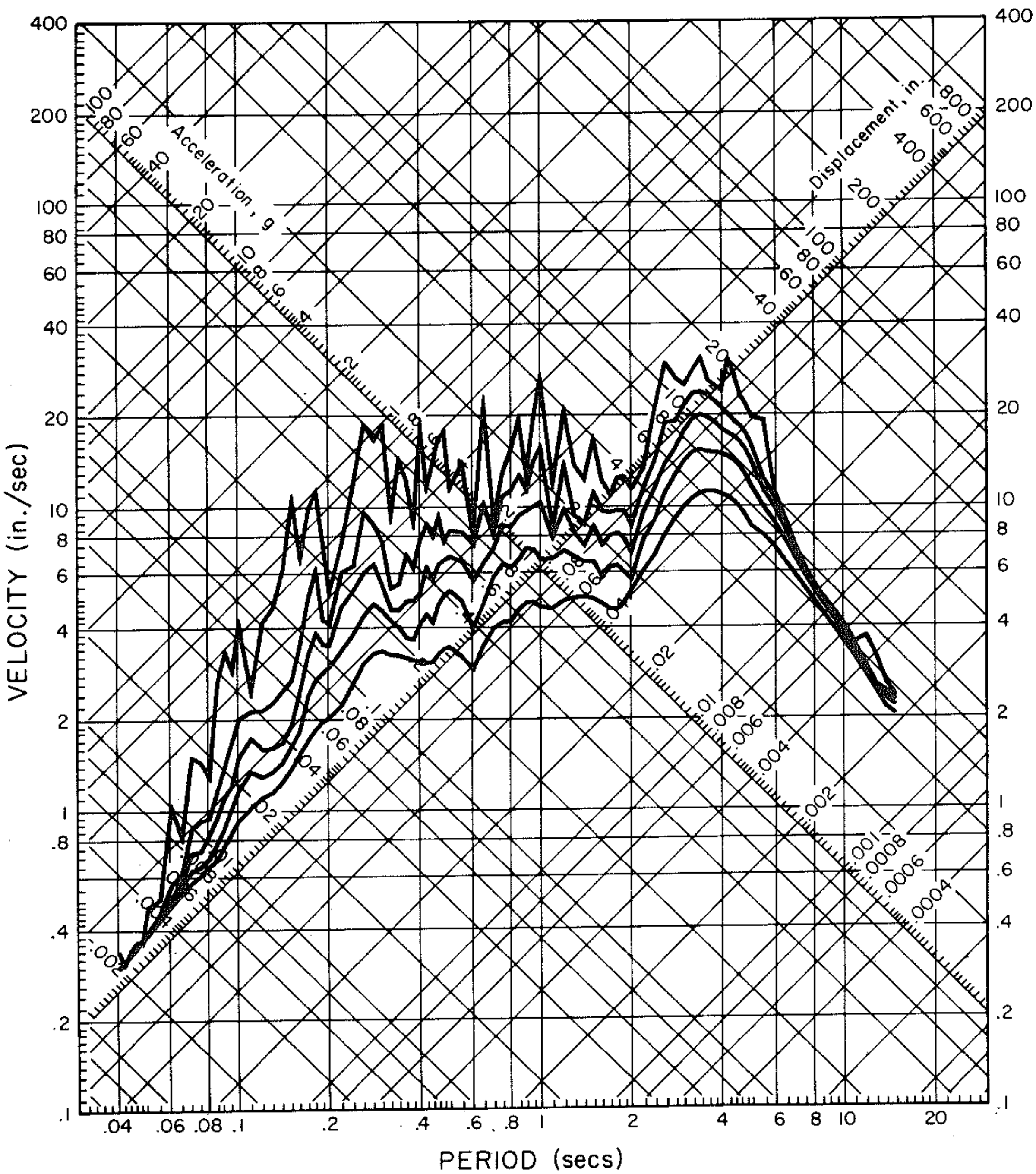
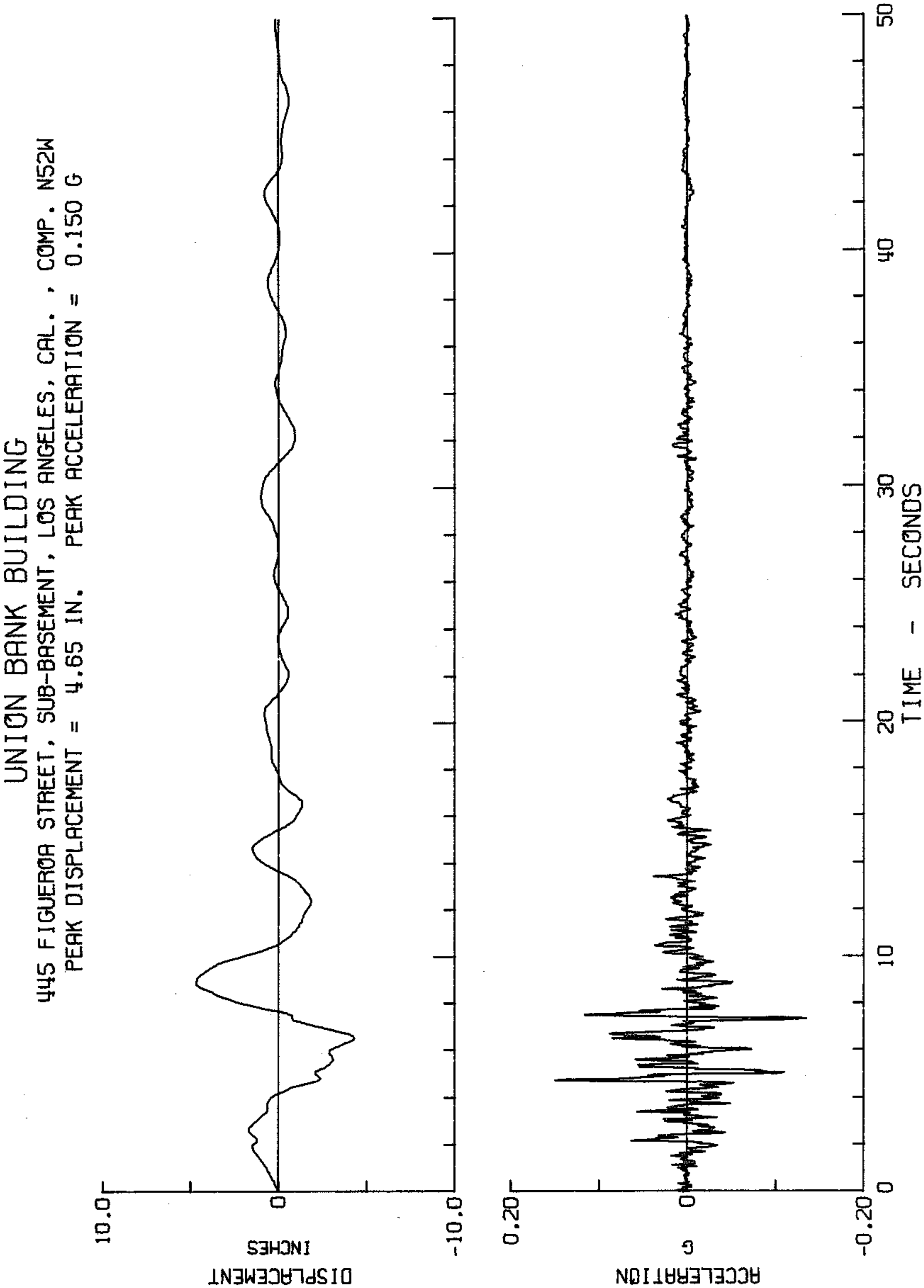


Figure 1.6



UNION BANK BUILDING
445 FIGUEROA STREET, SUB-BASEMENT, LOS ANGELES, CAL. , COMP. N52W
PEAK DISPLACEMENT = 4.65 IN. PEAK ACCELERATION = 0.150 G
DISPLACEMENT
INCHES
10.0
0
-10.0
ACCELERATION
G
0.20
0
-0.20
0
10
20
30
40
50
TIME - SECONDS

Figure 1.7

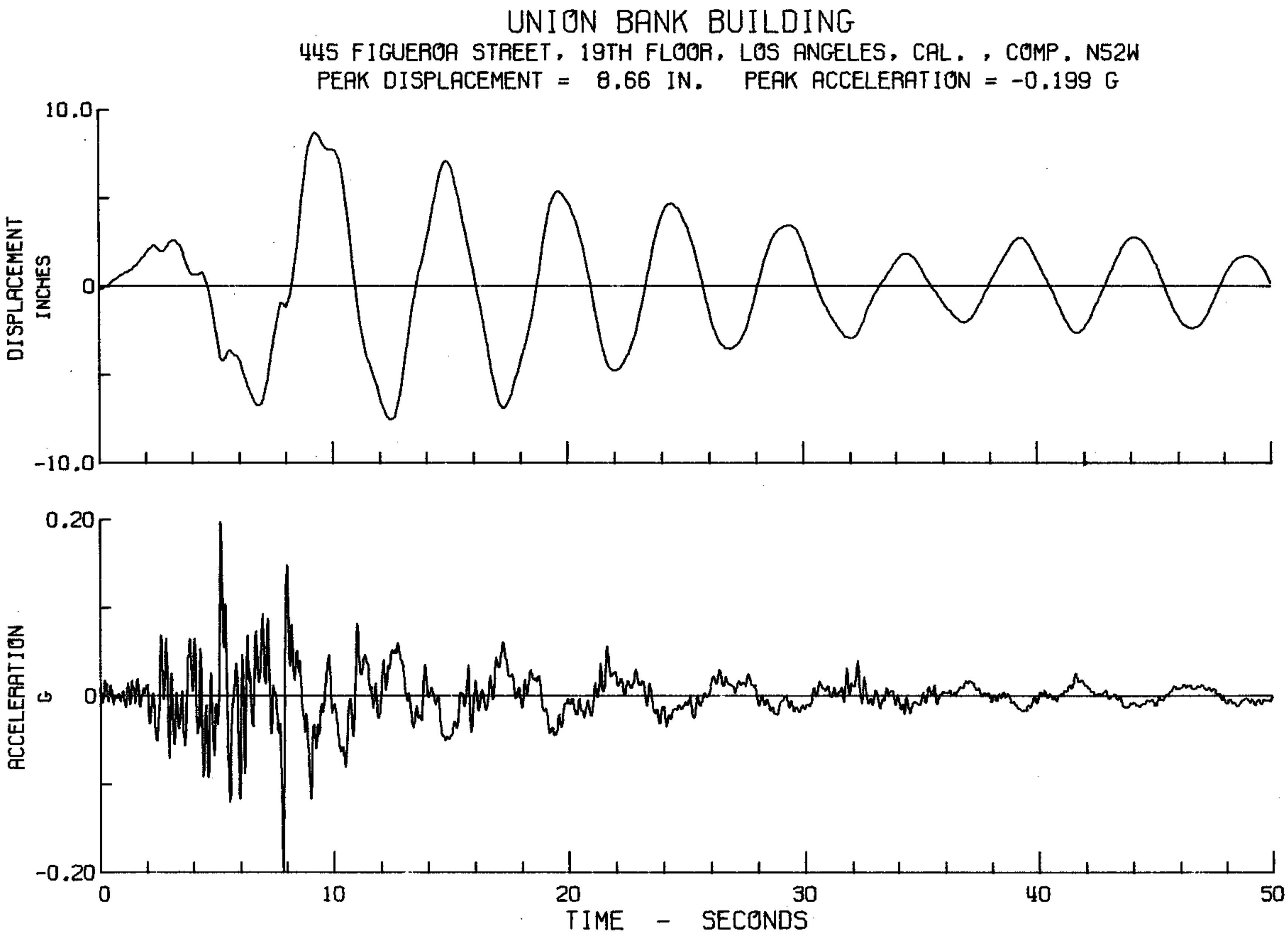

UNION BANK BUILDING
445 FIGUEROA STREET, 19TH FLOOR, LOS ANGELES, CAL. , COMP. N52W
PEAK DISPLACEMENT = 8.66 IN. PEAK ACCELERATION = -0.199 G
DISPLACEMENT INCHES
10.0
0
-10.0
ACCELERATION G
0.20
0
-0.20
0
10
20
30
40
50
TIME - SECONDS

Figure 1.8

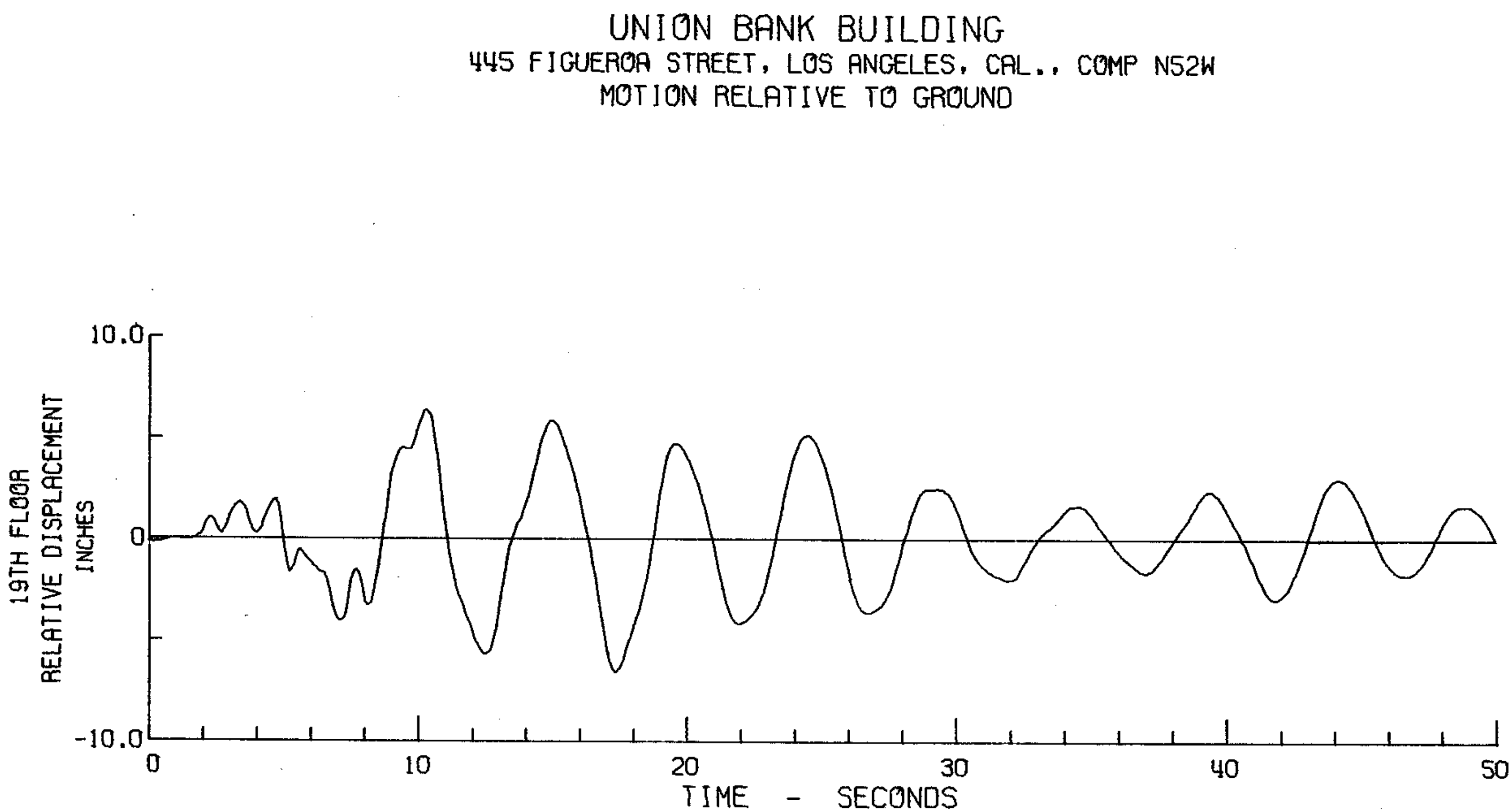

UNION BANK BUILDING
445 FIGUEROA STREET, LOS ANGELES, CAL., COMP N52W
MOTION RELATIVE TO GROUND
10.0
0
-10.0
19TH FLOOR
RELATIVE DISPLACEMENT
INCHES
0
10
20
30
40
50
TIME - SECONDS

Figure 1.9

RESPONSE SPECTRUM

UNION BANK BUILDING

445 FIGUEROA STREET, SUB-BASEMENT, LOS ANGELES, CAL. , COMP. N52W

DAMPING VALUES ARE 0, 2, 5, 10 AND 20 PERCENT OF CRITICAL

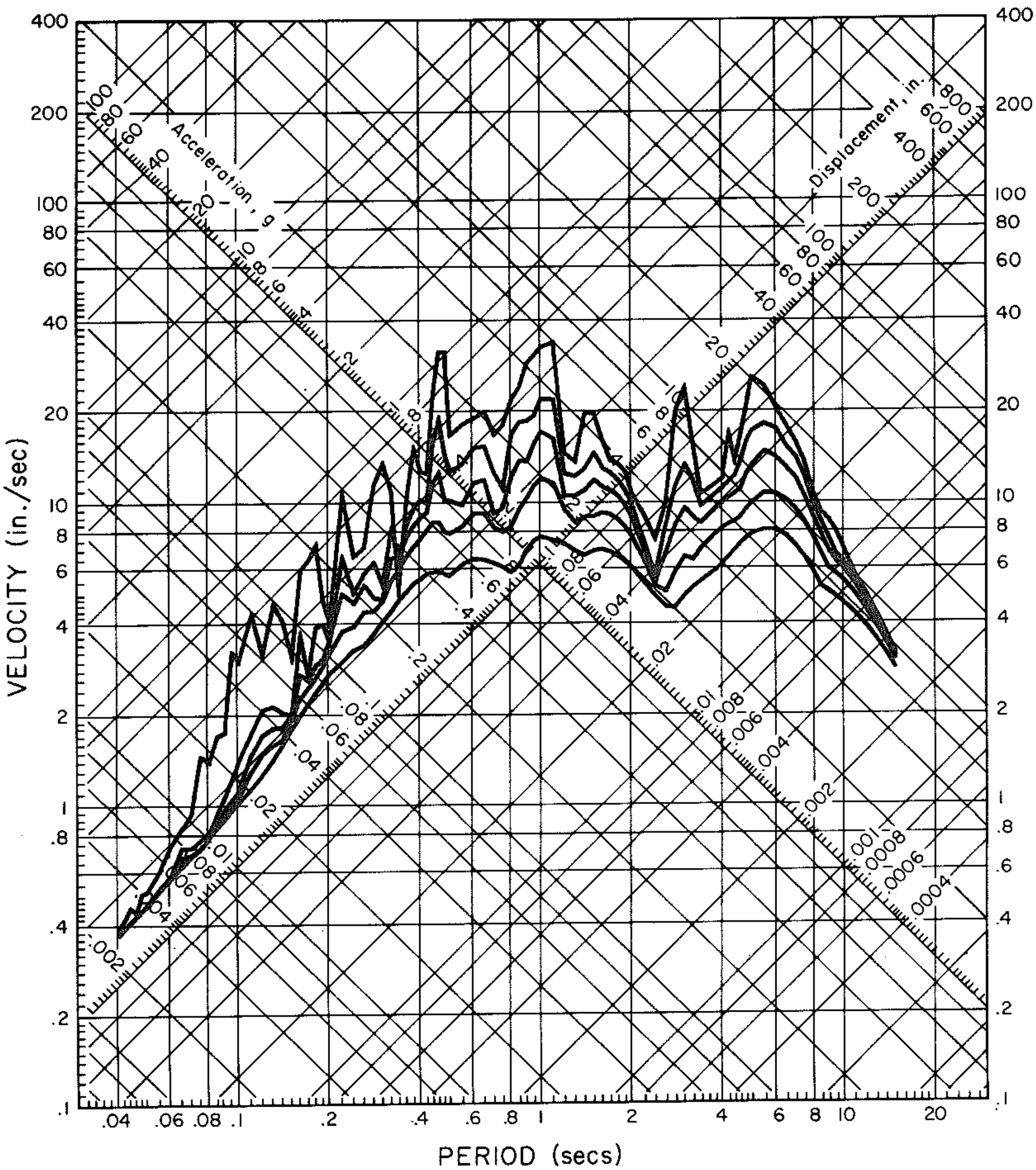


Figure 1.10

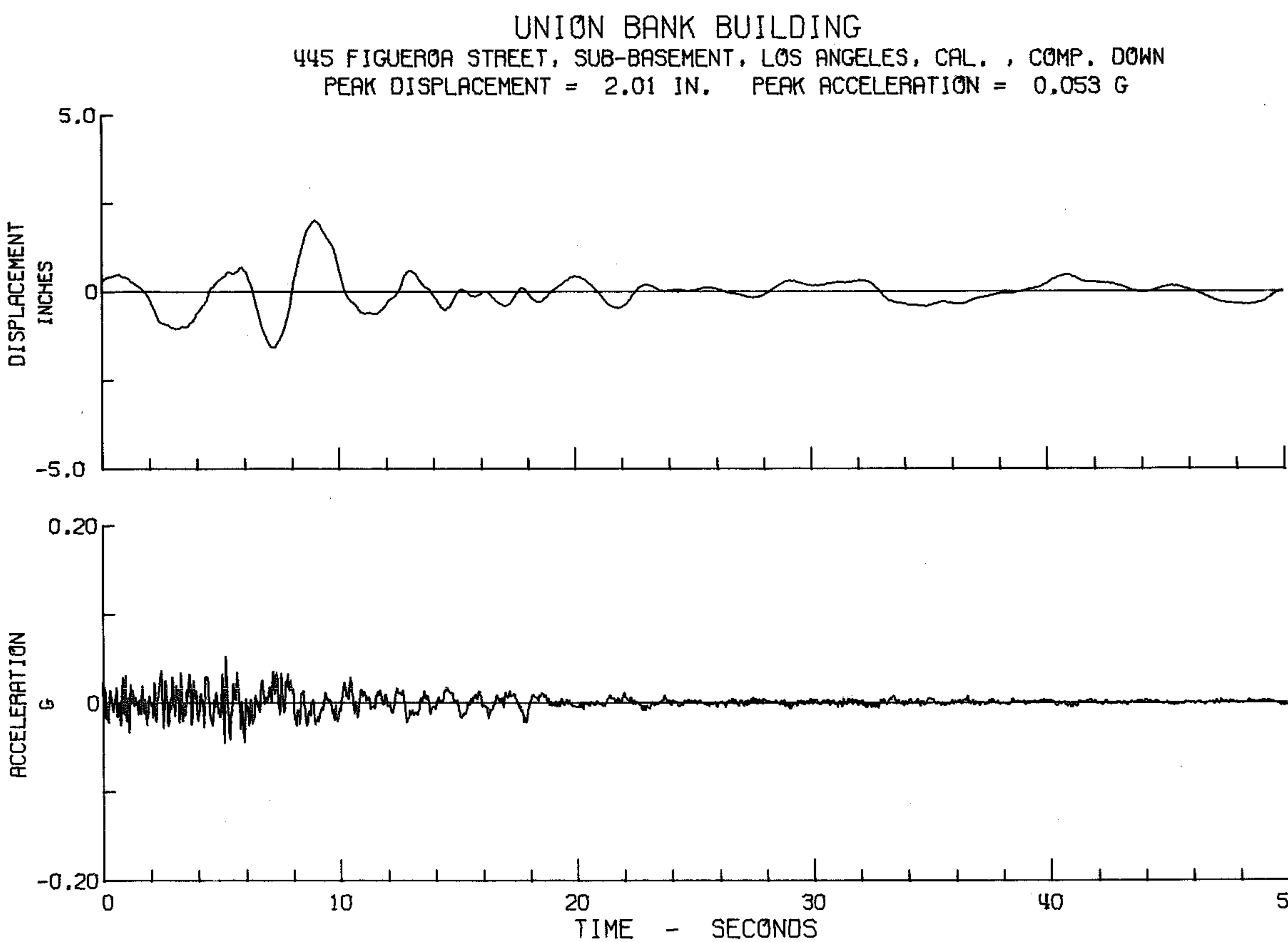

UNION BANK BUILDING
445 FIGUEROA STREET, SUB-BASEMENT, LOS ANGELES, CAL. , COMP. DOWN
PEAK DISPLACEMENT = 2.01 IN. PEAK ACCELERATION = 0.053 G
5.0
0
-5.0
DISPLACEMENT
INCHES
0.20
0
-0.20
ACCELERATION
G
0
10
20
30
40
50
TIME - SECONDS

Figure 1.11

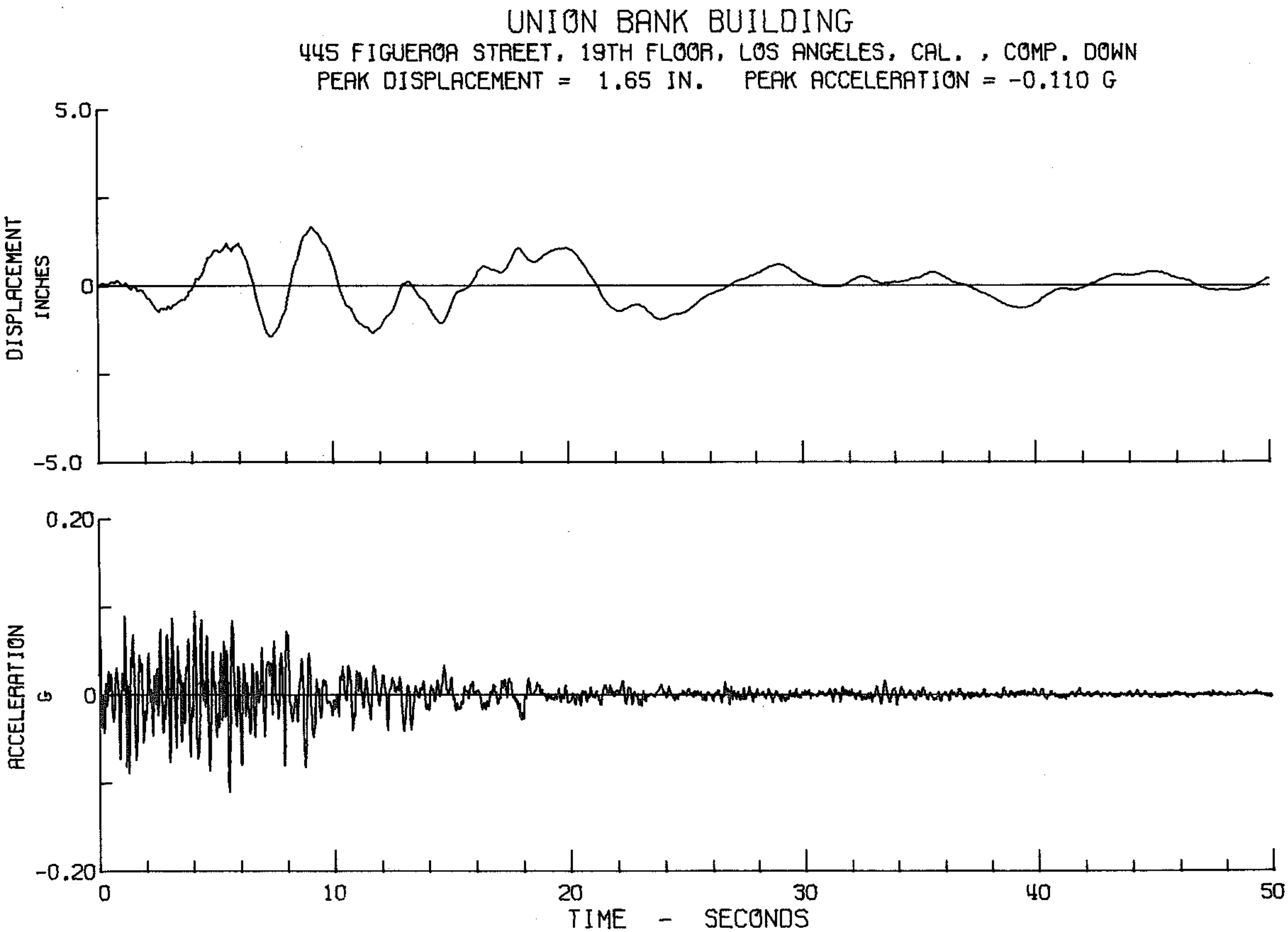

UNION BANK BUILDING
445 FIGUEROA STREET, 19TH FLOOR, LOS ANGELES, CAL. , COMP. DOWN
PEAK DISPLACEMENT = 1.65 IN. PEAK ACCELERATION = -0.110 G
DISPLACEMENT
INCHES
5.0
0
-5.0
ACCELERATION
G
0.20
0
-0.20
0
10
20
30
40
50
TIME - SECONDS

Figure 1.12

RESPONSE SPECTRUM

UNION BANK BUILDING

445 FIGUEROA STREET, SUB-BASEMENT, LOS ANGELES, CAL., COMP. DOWN

DAMPING VALUES ARE 0, 2, 5, 10 AND 20 PERCENT OF CRITICAL

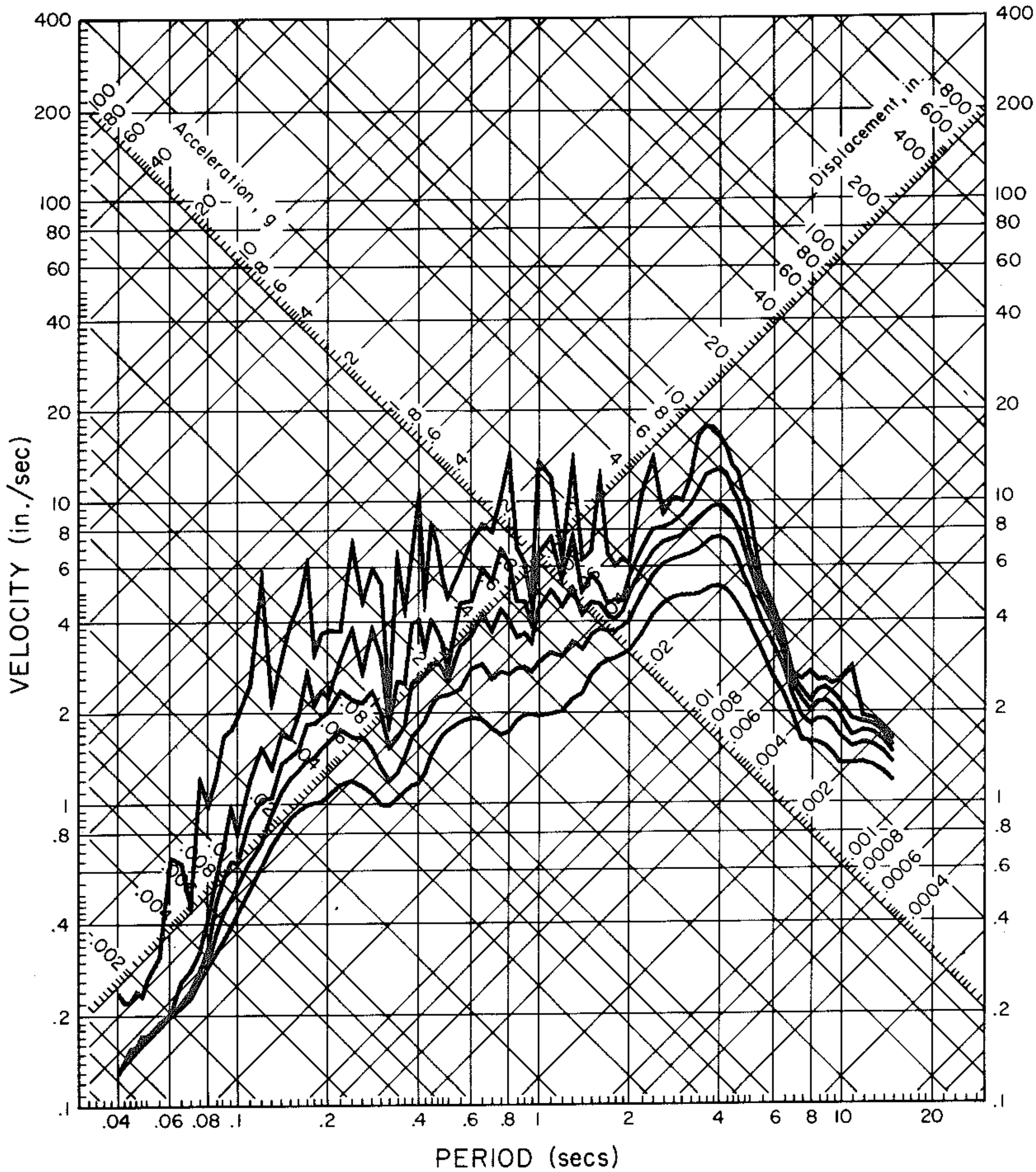


Figure 1.13

Chapter 2

Kajima International Building
250 E. First Street
Los Angeles, California

The Kajima International building, designed in 1966, is located in downtown Los Angeles approximately 21 miles south of the center of the San Fernando earthquake. The 15-story office tower is a steel frame building that measures 66 by 96 feet in plan and stands 202 feet above grade. The basement, 1st and 2nd floor areas are used primarily for retail and banking space while the 3rd through 14th floors provide office space. The entire 15th floor and the portion of the roof enclosed by the penthouse contain mechanical equipment. Figure 2. 1 is a picture of the northwest elevation of the Kajima International building. The building sustained no significant damage.

A three-dimensional moment resisting steel fame provides the resistance to both lateral and vertical loads. Four moment resisting frames are provided in both the transverse and longitudinal directions. Lightweight reinforced concrete floor slabs act as rigid diaphrams in the horizontal direction. Concrete encasement of the exterior columns of the frame, designed for fire protection, provide additional stiffness. Precast concrete spandrels, 6 feet deep, are used as part of the exterior facia of the building, and they also provide additional stiffness at low levels of vibration. The foundation system consists of spread footings combined in pairs. Although a seismic gap was provided between the main tower and an adjacent three-story parking facility, apparently impacting occurred during the earthquake. Figures 2. 2a and 2. 2b are schematic drawings of a transverse section and a floor plan of the building.

Strong-motion instruments located at the basement, seventh floor, and roof of the building recorded two horizontal and one vertical record of acceleration each. Plots of these accelerations and the integrated displacements, together with the response spectra for the three components of recorded basement motion, are shown in the following figures.

It can be seen in the seismograms that the Kajima building was strongly excited by a prominent displacement pulse just as happened in the case of the Union Bank building. It can also be seen from the seismograms that the accelerographs were slow in starting to record and, therefore, some of the ground and building motion was lost from the record. The horizontal displacement records indicate that the fundamental period of vibration in both directions was approximately 2.9 seconds during the earthquake. Pre-earthquake and post-earthquake ambient tests [4] indicated that the fundamental period in the N36^{o}E direction was 1.3 second and 2.1 seconds respectively. This change in period is similar to that observed for the Union Bank building and is assumed to result from the degradation of stiffness of nonstructural elements. A similar phenomenon was observed for the N54^{o}W motions.

The records of the vertical motion on the roof of the Kajima building show that the accelerations were produced mainly by high frequency vibrations of the structural frame, whereas, the vertical displacement of the building resulted mainly from the building riding the vertical ground displacements as a ship rides the waves.

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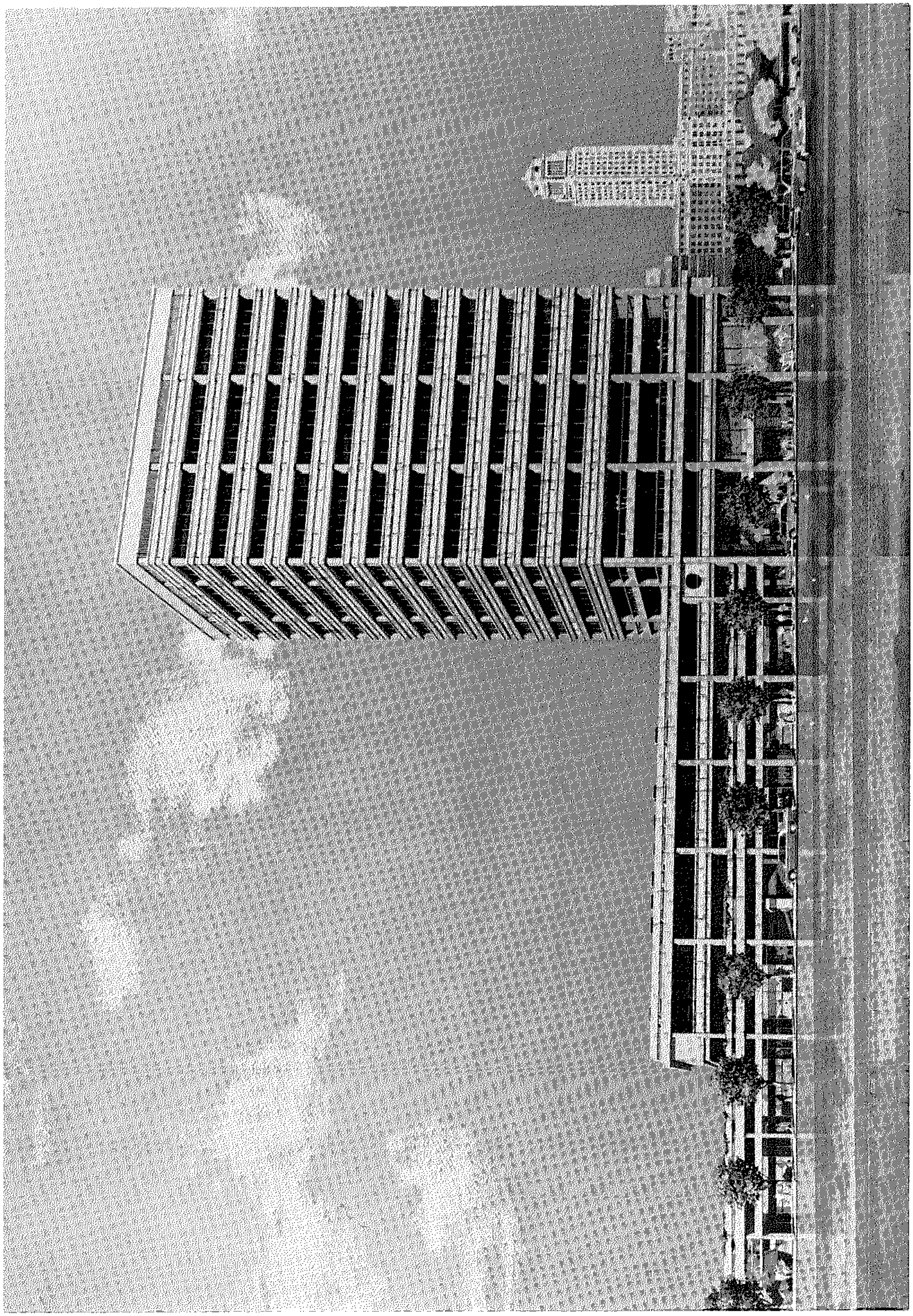


Figure 2.1 Kajima International Building

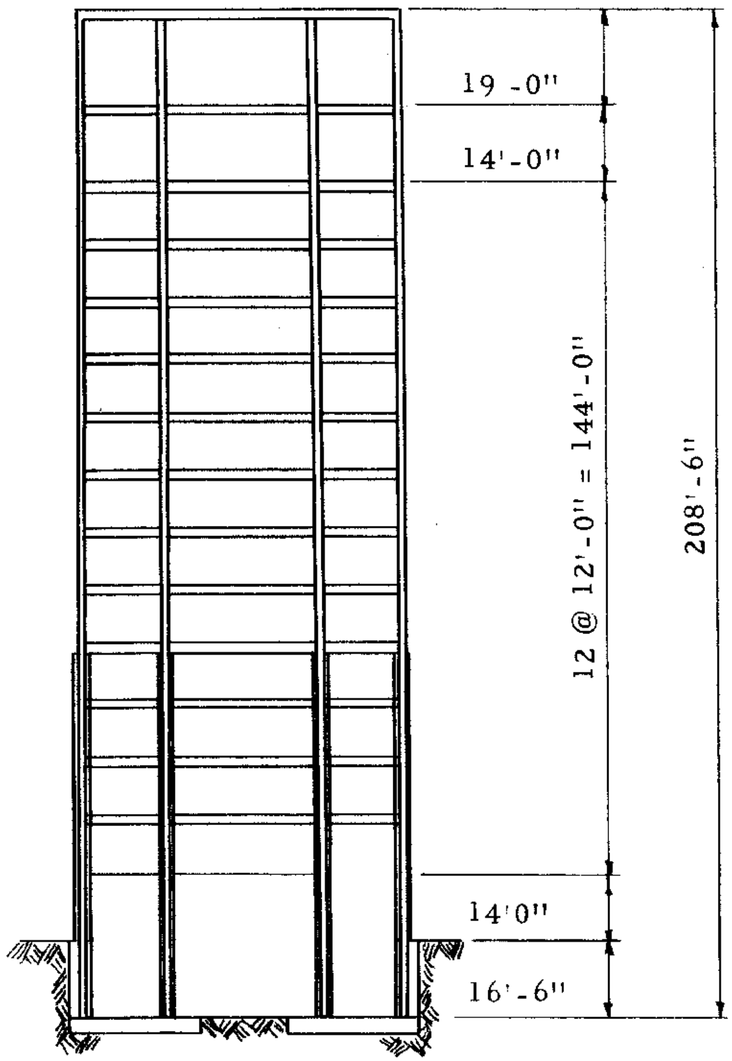


Figure 2. 2a Transverse Section

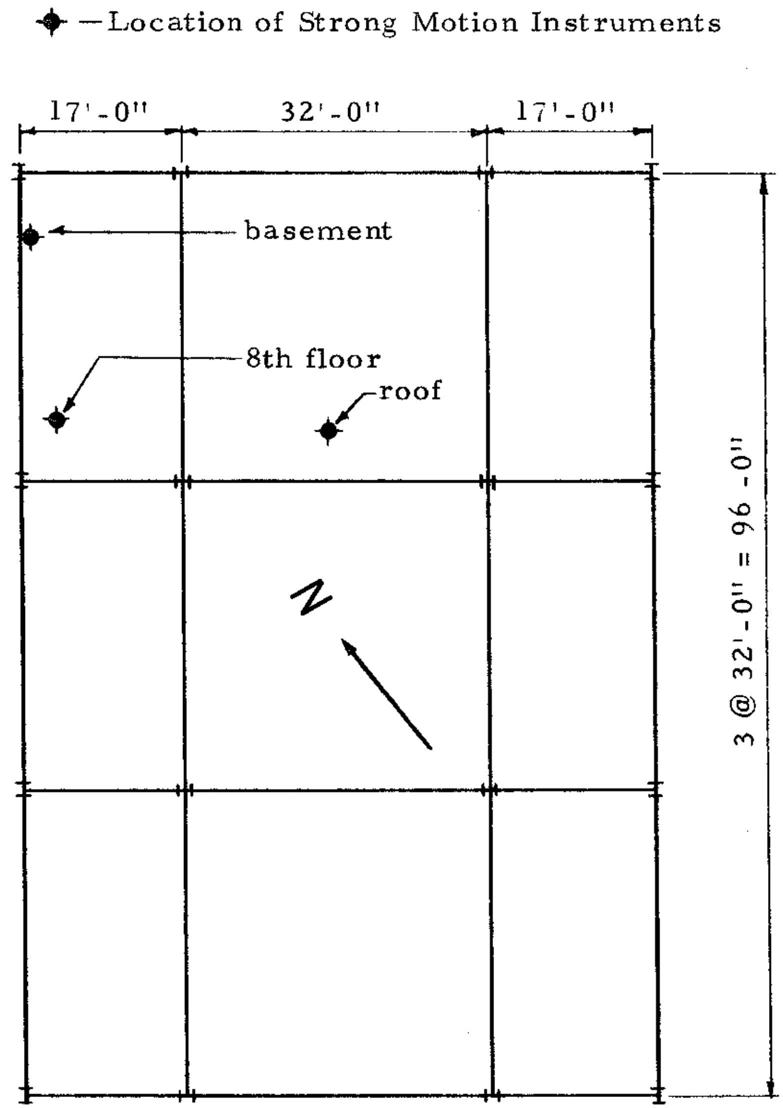


Figure 2. 2b Typical Floor Plan

Figure 2. 2 Schematic of Kajima International Building Structural System

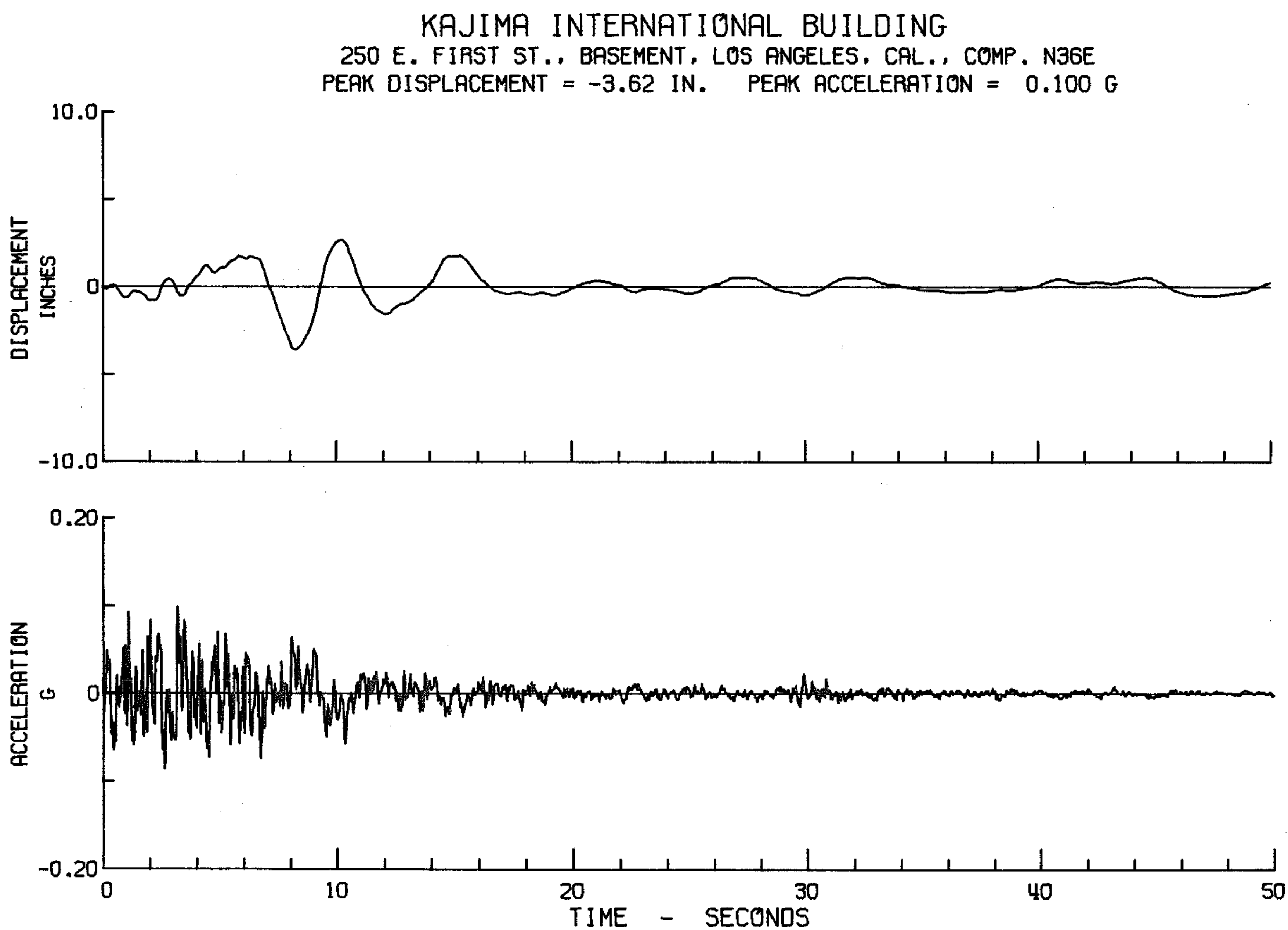

KAJIMA INTERNATIONAL BUILDING
250 E. FIRST ST., BASEMENT, LOS ANGELES, CAL., COMP. N36E
PEAK DISPLACEMENT = -3.62 IN.   PEAK ACCELERATION = 0.100 G
DISPLACEMENT
INCHES
10.0
0
-10.0
ACCELERATION
G
0.20
0
-0.20
0
10
20
30
40
50
TIME - SECONDS

Figure 2.3

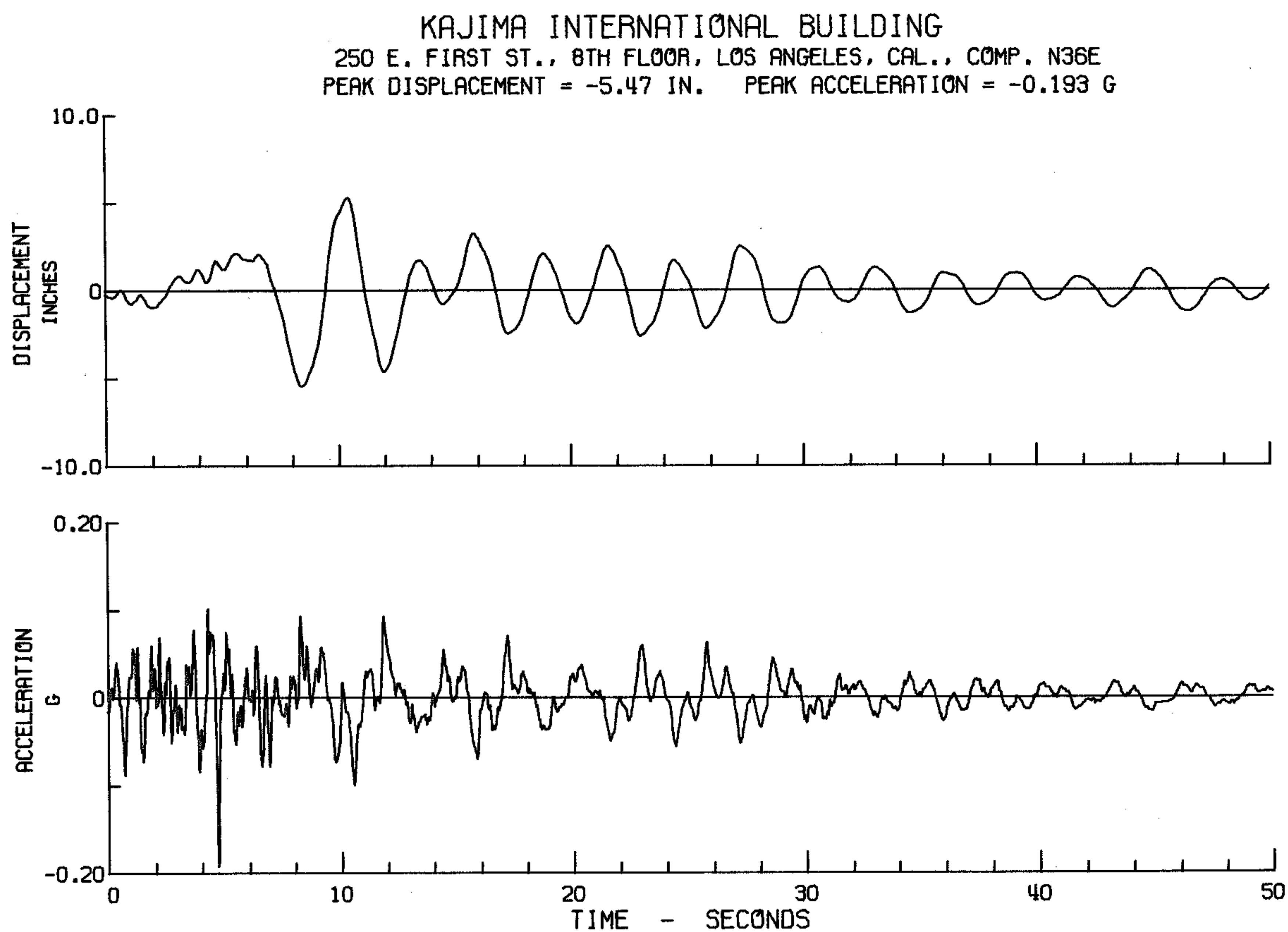

KAJIMA INTERNATIONAL BUILDING
250 E. FIRST ST., 8TH FLOOR, LOS ANGELES, CAL., COMP. N36E
PEAK DISPLACEMENT = -5.47 IN.   PEAK ACCELERATION = -0.193 G
10.0
0
-10.0
DISPLACEMENT
INCHES
0.20
0
-0.20
ACCELERATION
G
0
10
20
30
40
50
TIME - SECONDS

Figure 2.4

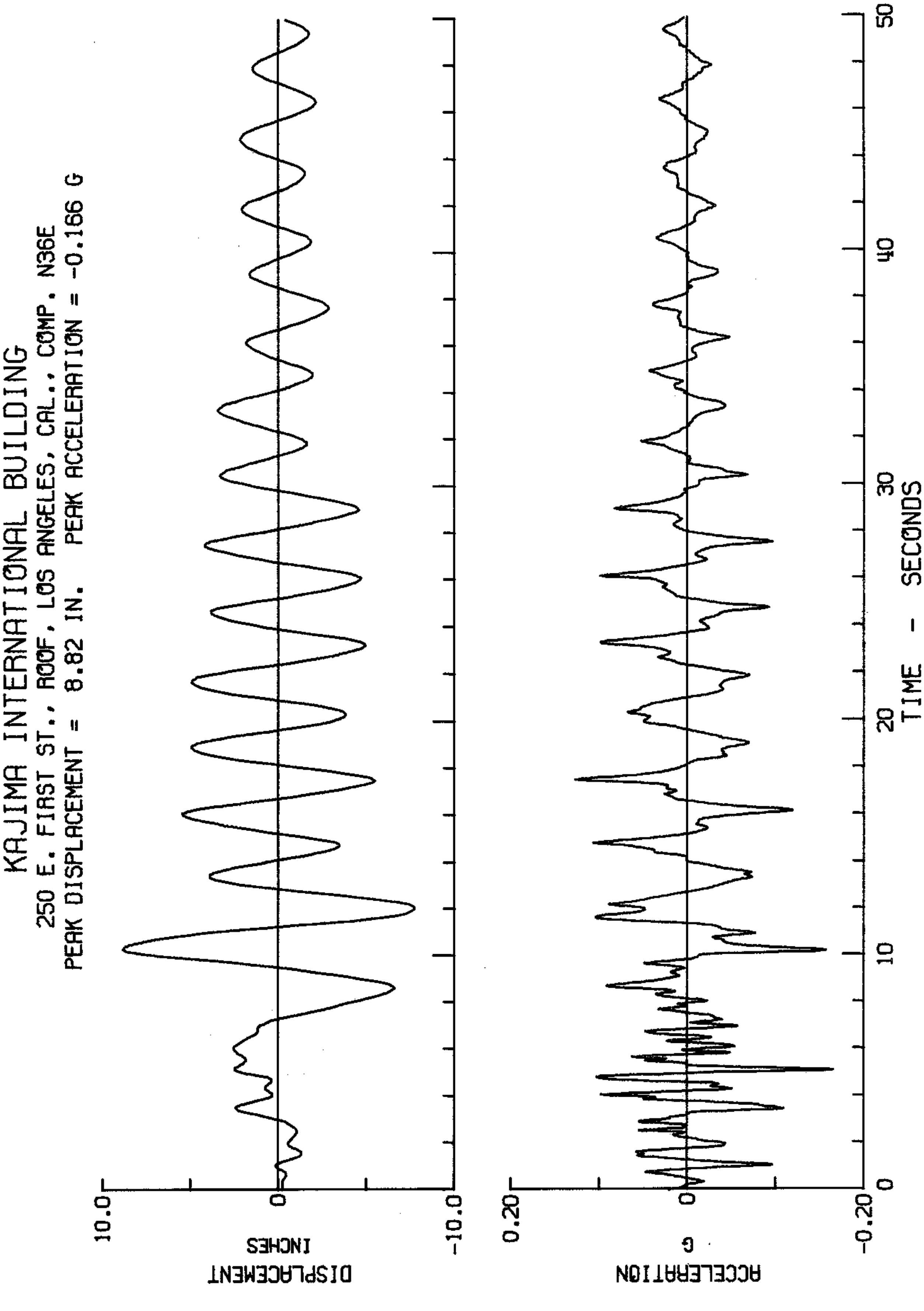

KAJIMA INTERNATIONAL BUILDING
250 E. FIRST ST., ROOF, LOS ANGELES, CAL., COMP. N36E
PEAK DISPLACEMENT = 8.82 IN.   PEAK ACCELERATION = -0.166 G
DISPLACEMENT INCHES
10.0
0
-10.0
ACCELERATION G
0.20
0
-0.20
0
10
20
30
40
50
TIME - SECONDS

Figure 2.5

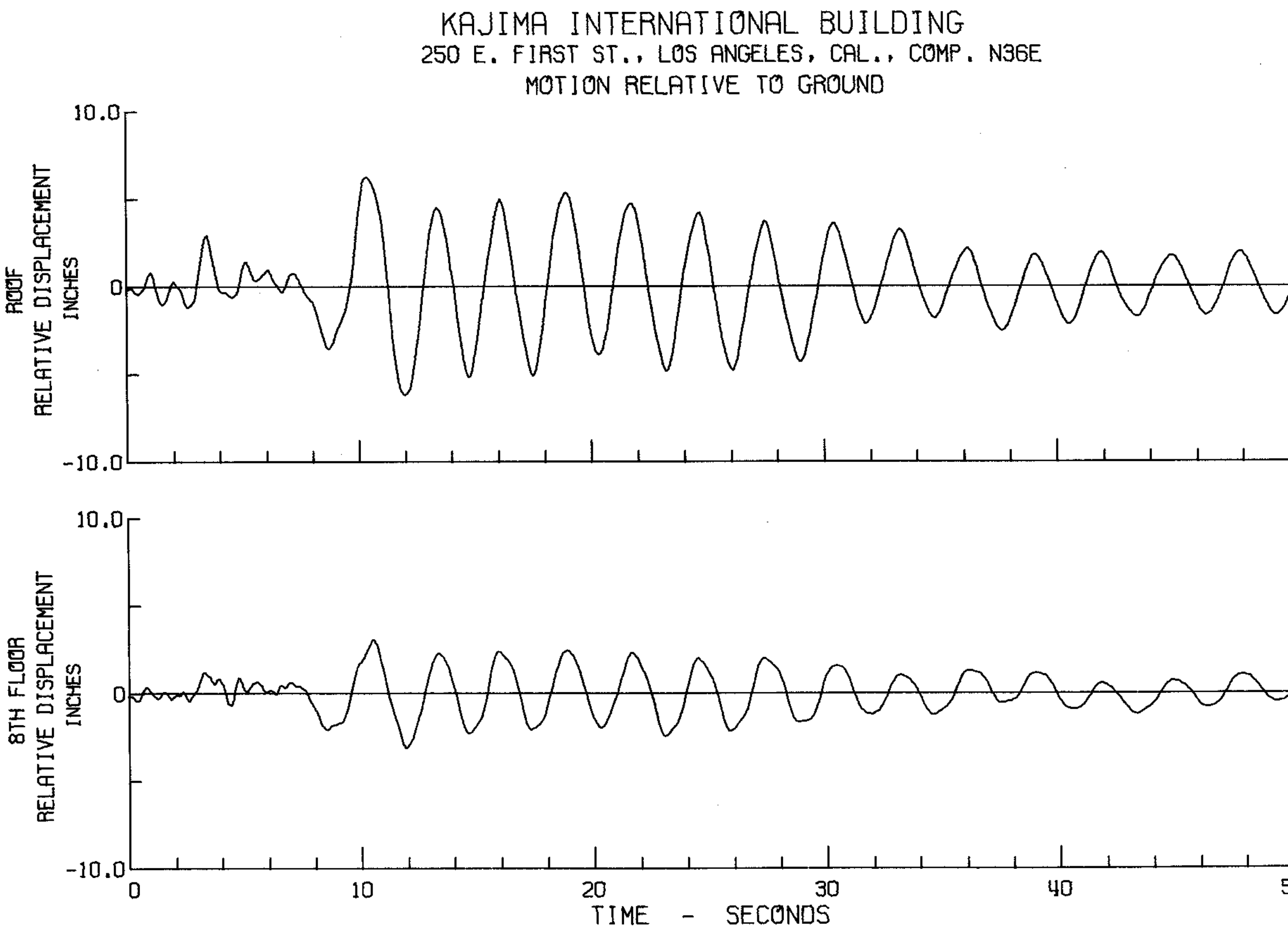

KAJIMA INTERNATIONAL BUILDING
250 E. FIRST ST., LOS ANGELES, CAL., COMP. N36E
MOTION RELATIVE TO GROUND
ROOF
RELATIVE DISPLACEMENT
INCHES
10.0
0
-10.0
8TH FLOOR
RELATIVE DISPLACEMENT
INCHES
10.0
0
-10.0
0
10
20
30
40
50
TIME - SECONDS

Figure 2.6

RESPONSE SPECTRUM

KAJIMA INTERNATIONAL BUILDING

250 E. FIRST ST., BASEMENT, LOS ANGELES, CAL., COMP. N36E

DAMPING VALUES ARE 0, 2, 5, 10 AND 20 PERCENT OF CRITICAL

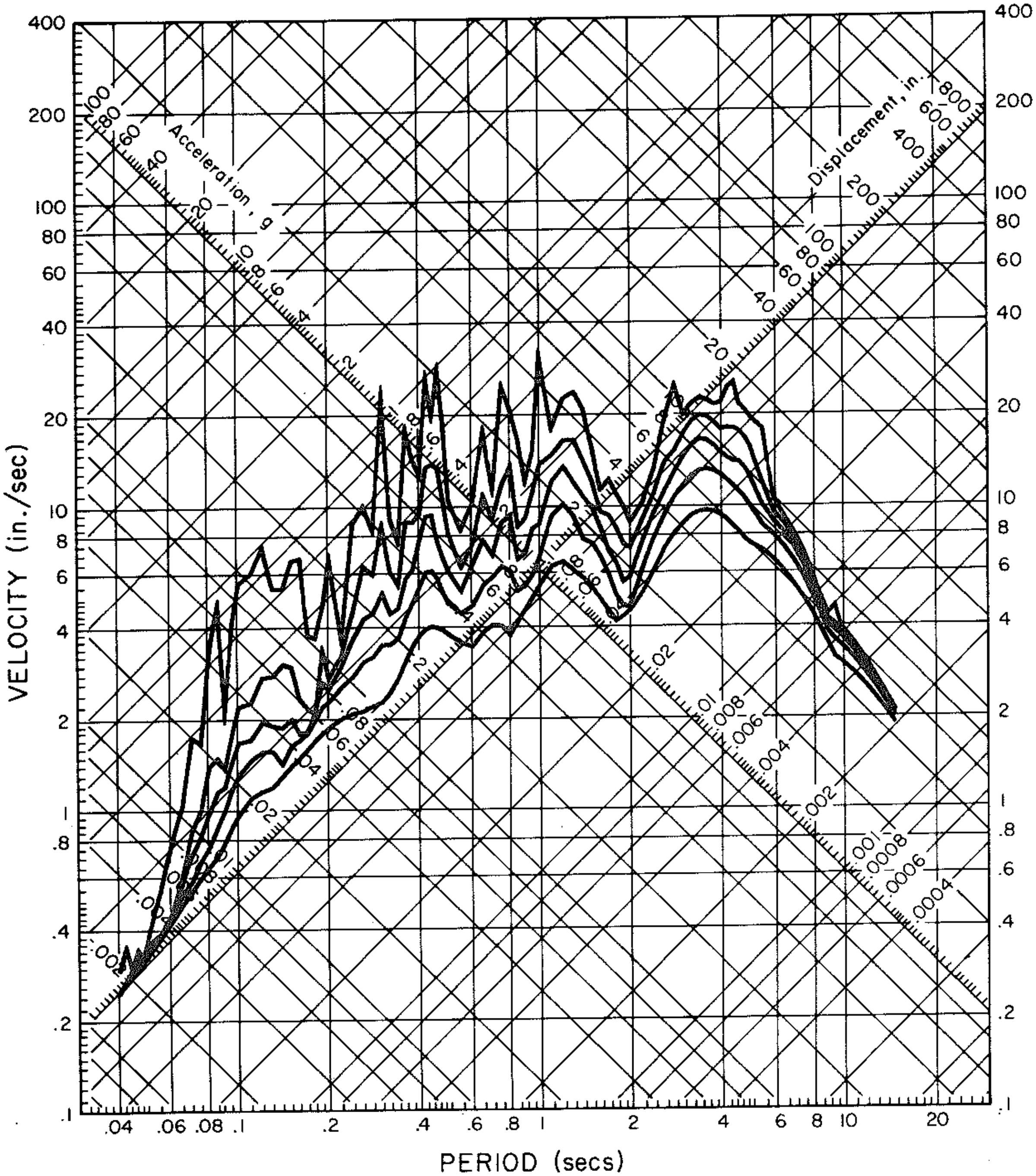


Figure 2.7

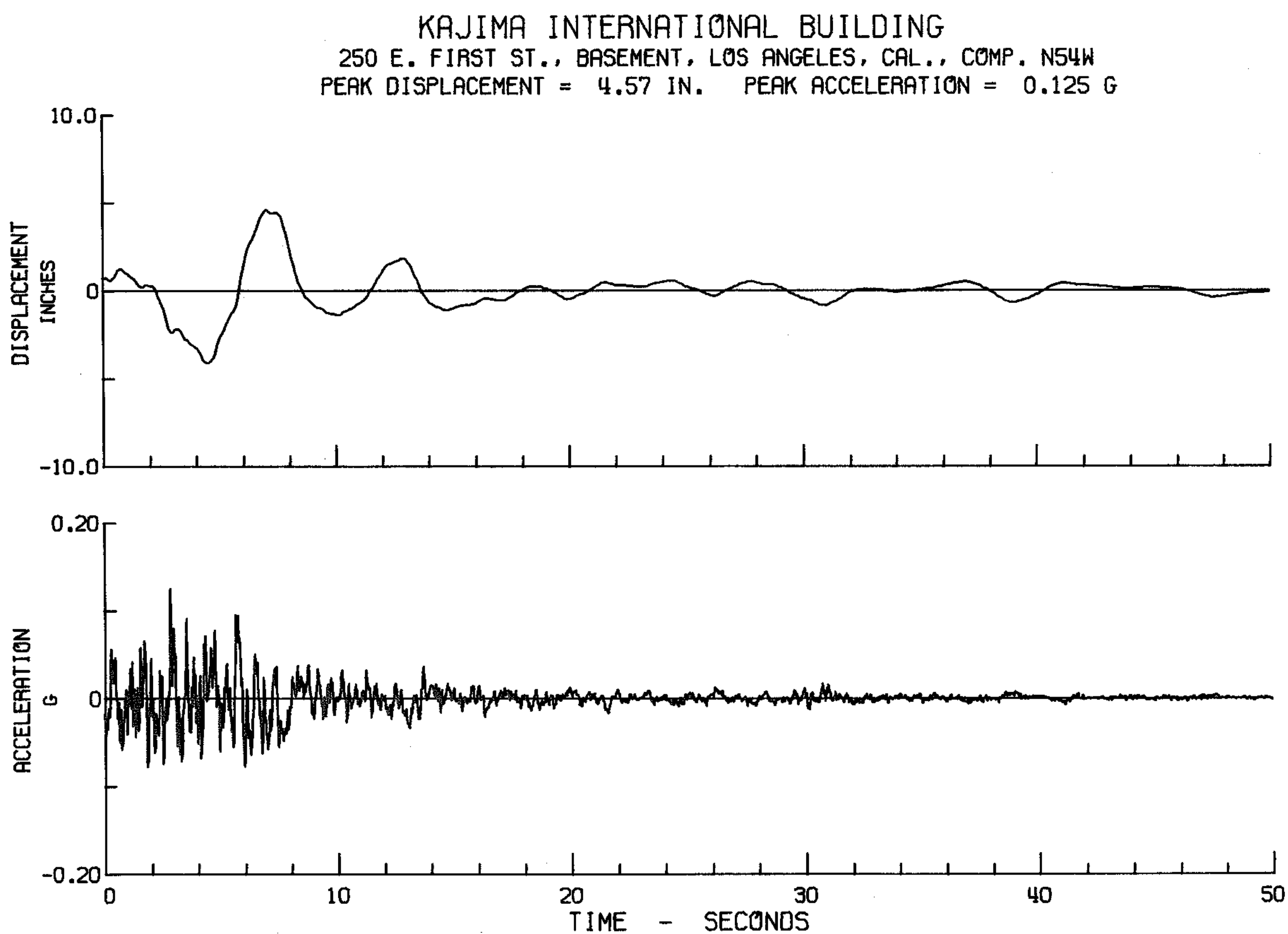

KAJIMA INTERNATIONAL BUILDING
250 E. FIRST ST., BASEMENT, LOS ANGELES, CAL., COMP. N54W
PEAK DISPLACEMENT = 4.57 IN. PEAK ACCELERATION = 0.125 G
10.0
0
-10.0
DISPLACEMENT
INCHES
0.20
0
-0.20
ACCELERATION
G
0
10
20
30
40
50
TIME - SECONDS

Figure 2.8

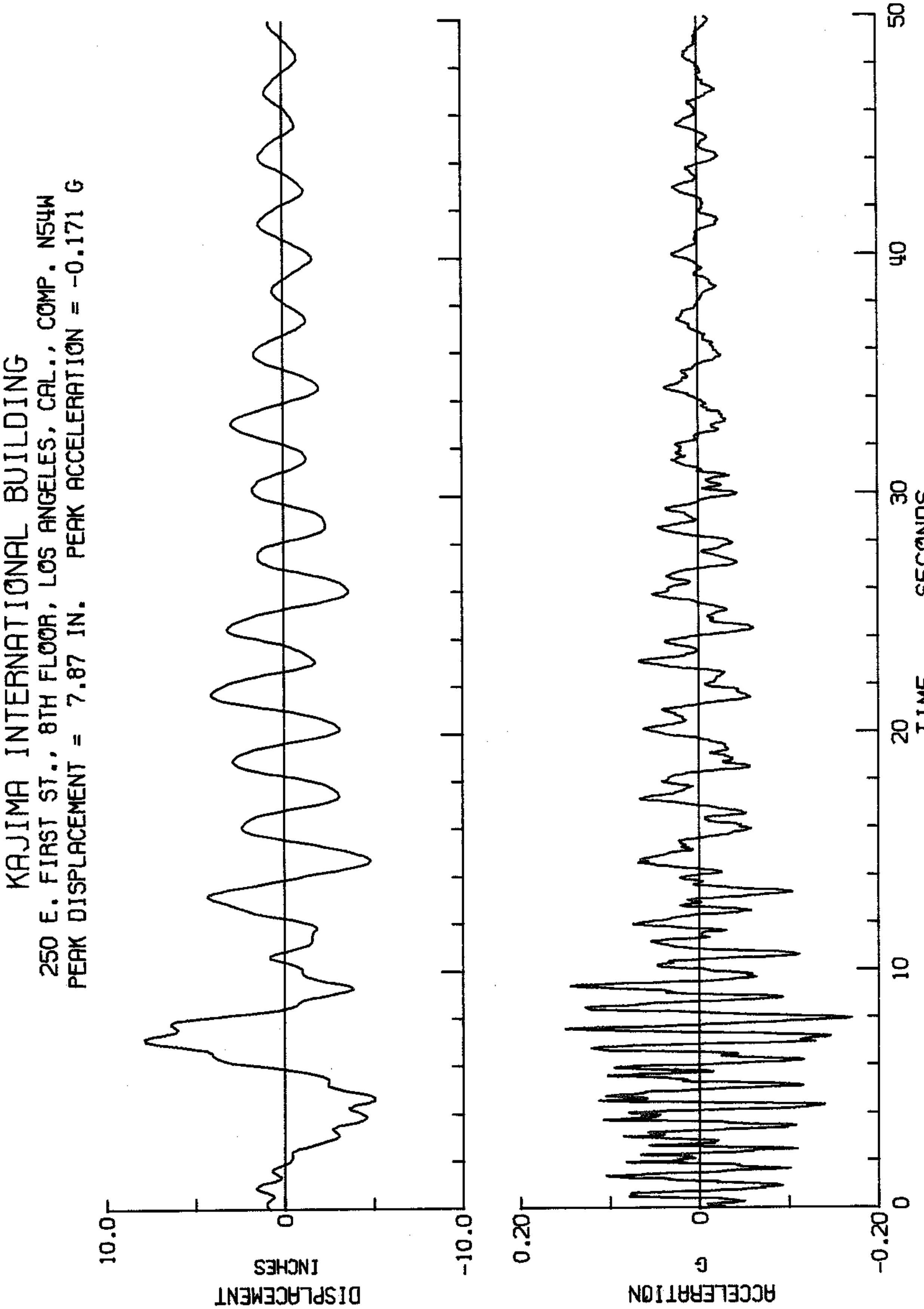

KAJIMA INTERNATIONAL BUILDING
250 E. FIRST ST., 8TH FLOOR, LOS ANGELES, CAL., COMP. N54W
PEAK DISPLACEMENT = 7.87 IN. PEAK ACCELERATION = -0.171 G
DISPLACEMENT
INCHES
10.0
0
-10.0
ACCELERATION
G
0.20
0
-0.20
0
10
20
30
40
50
TIME - SECONDS

Figure 2.9

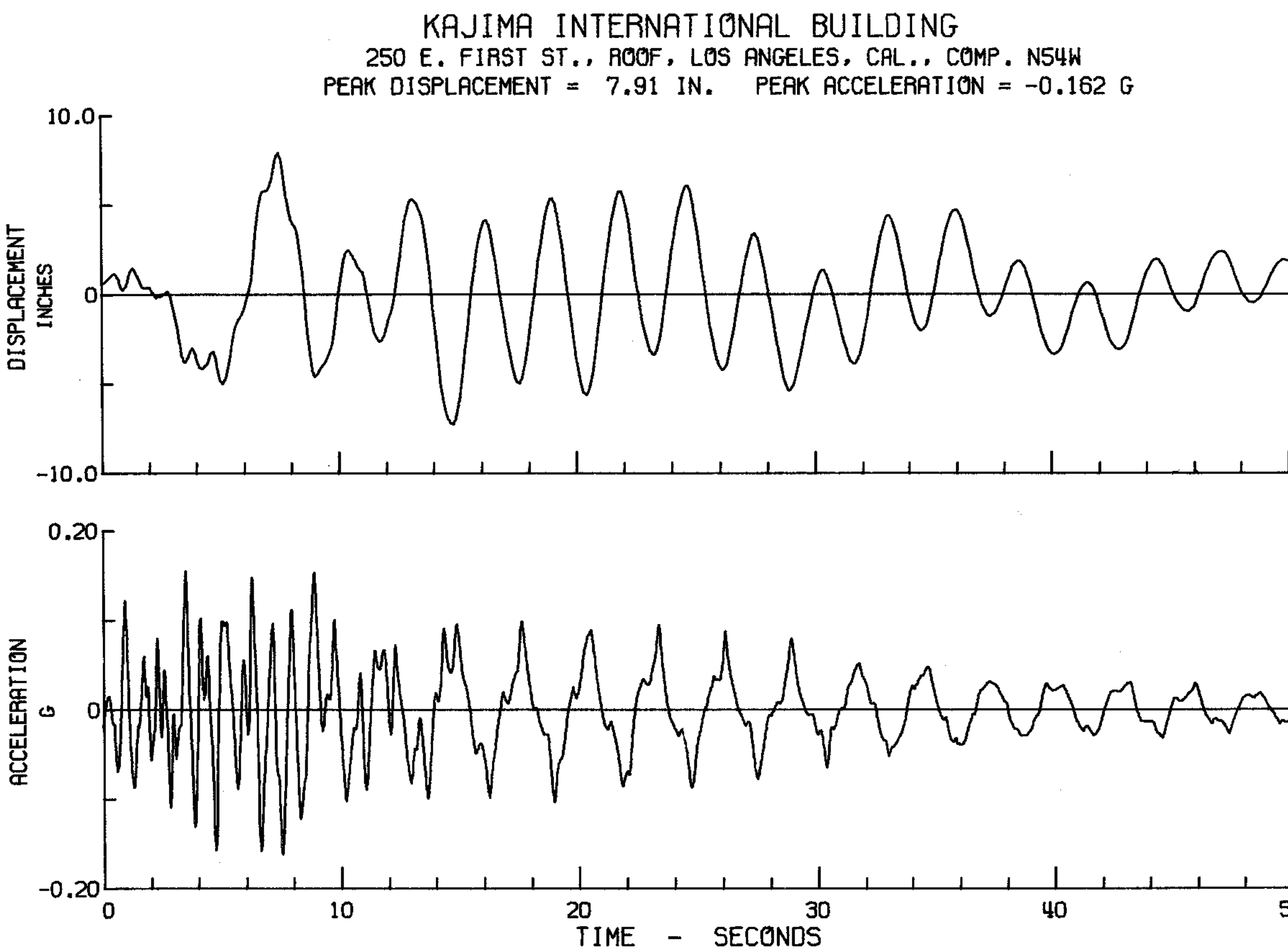

KAJIMA INTERNATIONAL BUILDING
250 E. FIRST ST., ROOF, LOS ANGELES, CAL., COMP. N54W
PEAK DISPLACEMENT = 7.91 IN. PEAK ACCELERATION = -0.162 G
DISPLACEMENT
INCHES
10.0
0
-10.0
ACCELERATION
G
0.20
0
-0.20
0
10
20
30
40
50
TIME - SECONDS

Figure 2.10

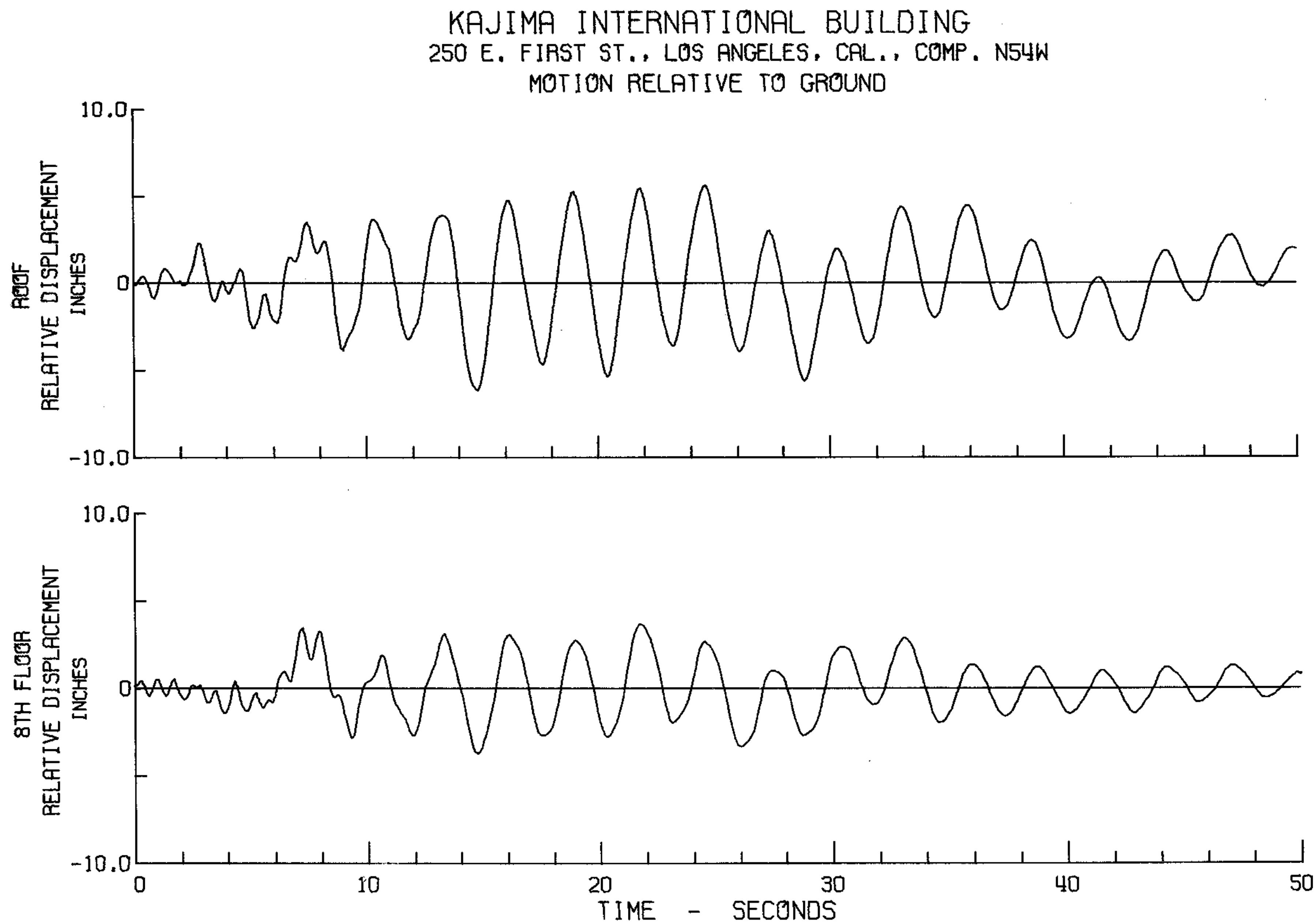

KAJIMA INTERNATIONAL BUILDING
250 E. FIRST ST., LOS ANGELES, CAL., COMP. N54W
MOTION RELATIVE TO GROUND
ROOF
RELATIVE DISPLACEMENT
INCHES
10.0
0
-10.0
8TH FLOOR
RELATIVE DISPLACEMENT
INCHES
10.0
0
-10.0
0
10
20
30
40
50
TIME - SECONDS

Figure 2.11

RESPONSE SPECTRUM

KAJIMA INTERNATIONAL BUILDING

250 E. FIRST ST., BASEMENT, LOS ANGELES, CAL., COMP. N54W

DAMPING VALUES ARE 0, 2, 5, 10 AND 20 PERCENT OF CRITICAL

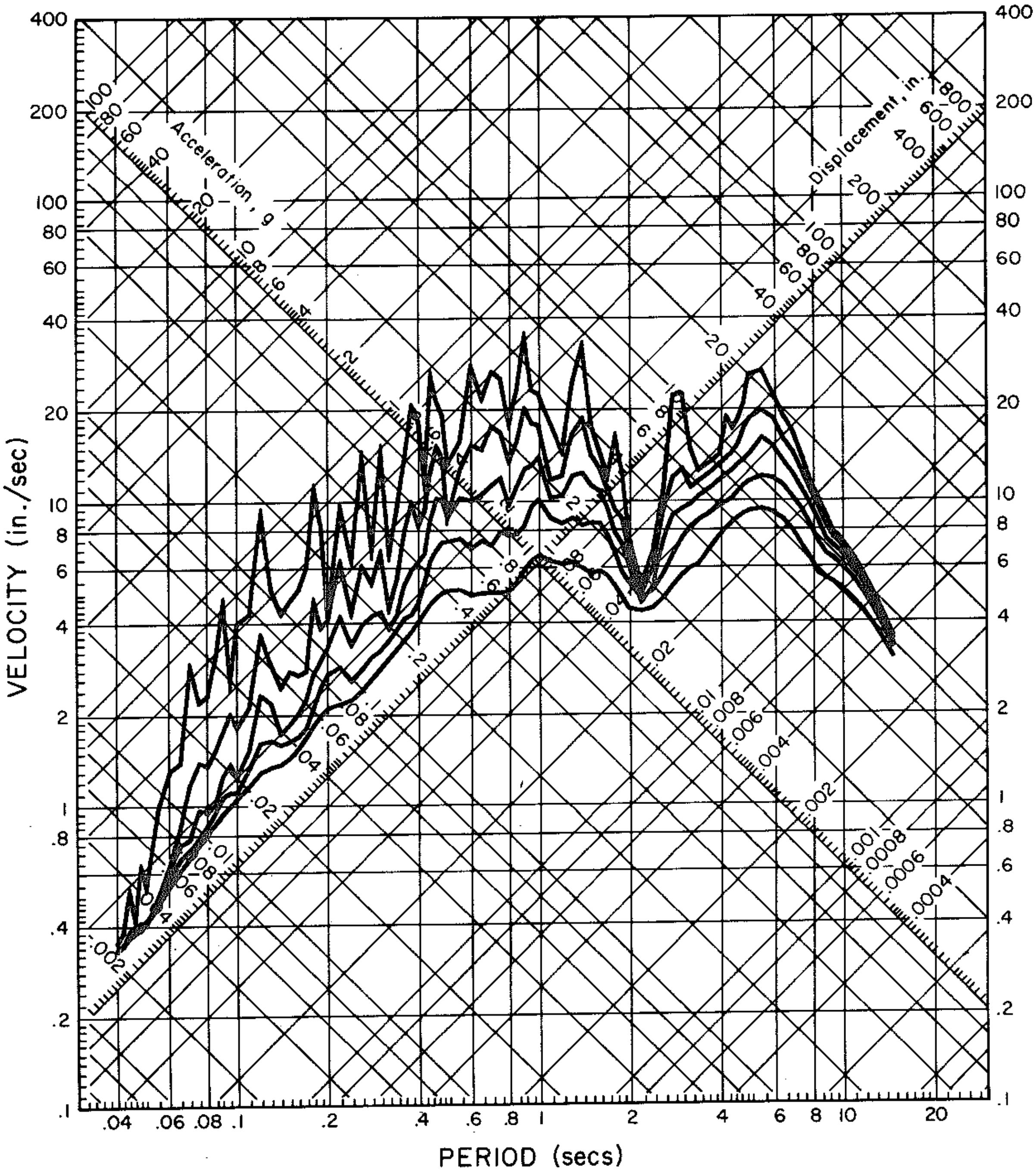


Figure 2.12

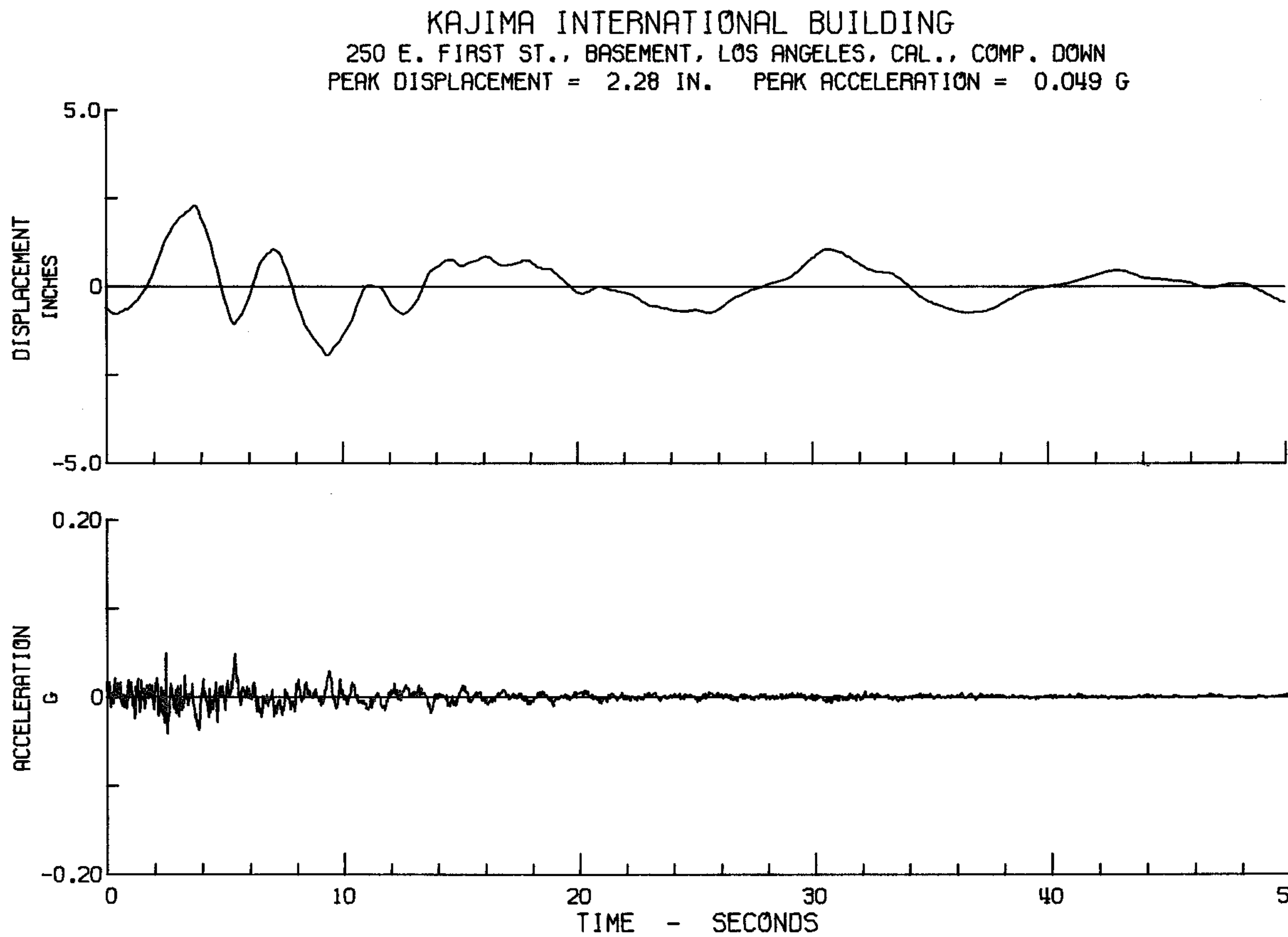

KAJIMA INTERNATIONAL BUILDING
250 E. FIRST ST., BASEMENT, LOS ANGELES, CAL., COMP. DOWN
PEAK DISPLACEMENT = 2.28 IN. PEAK ACCELERATION = 0.049 G
5.0
0
-5.0
DISPLACEMENT
INCHES
0.20
0
-0.20
ACCELERATION
G
0
10
20
30
40
50
TIME - SECONDS

Figure 2.13

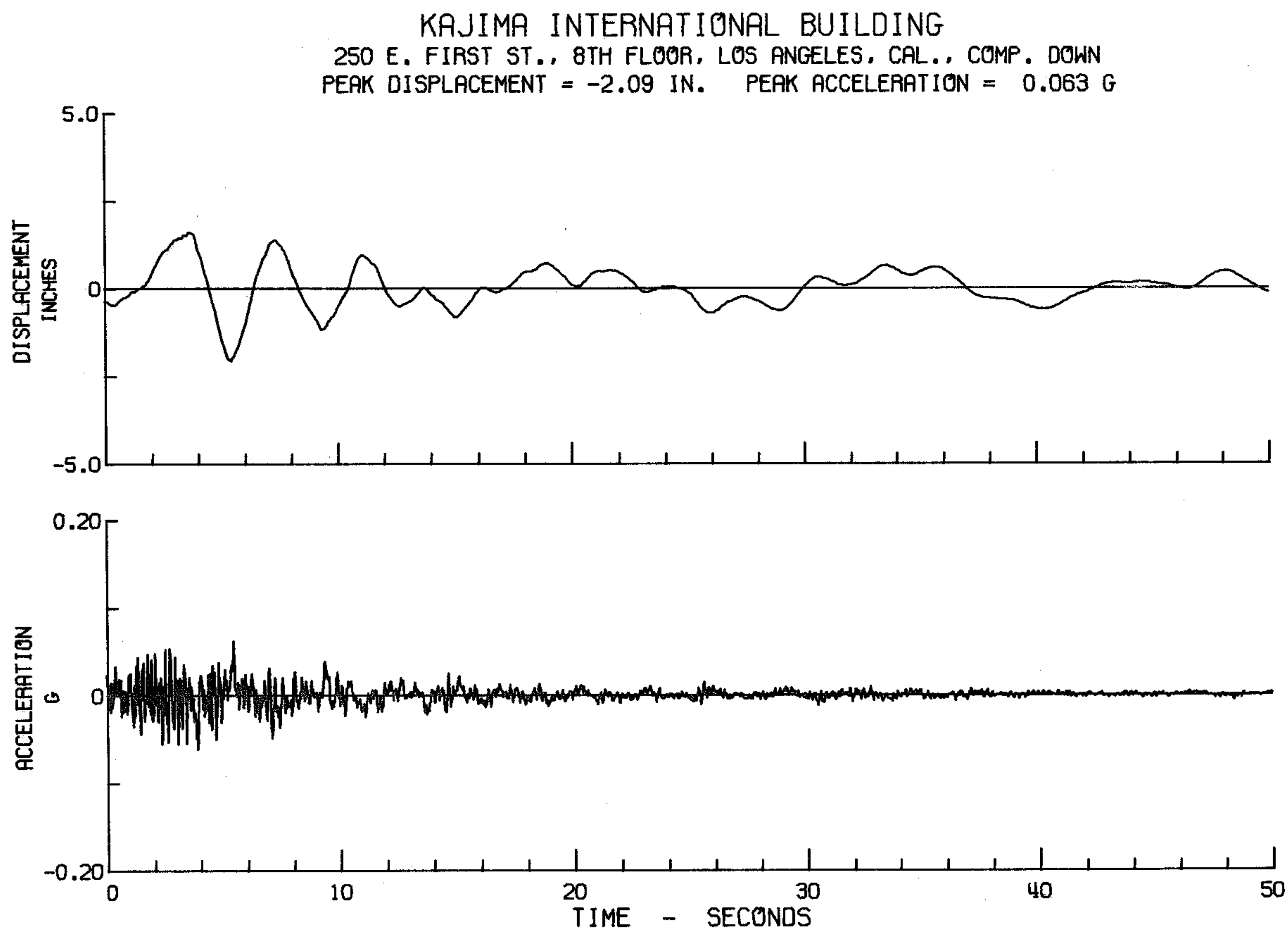

KAJIMA INTERNATIONAL BUILDING
250 E. FIRST ST., 8TH FLOOR, LOS ANGELES, CAL., COMP. DOWN
PEAK DISPLACEMENT = -2.09 IN.   PEAK ACCELERATION = 0.063 G
DISPLACEMENT
INCHES
5.0
0
-5.0
ACCELERATION
G
0.20
0
-0.20
0
10
20
30
40
50
TIME - SECONDS

Figure 2.14

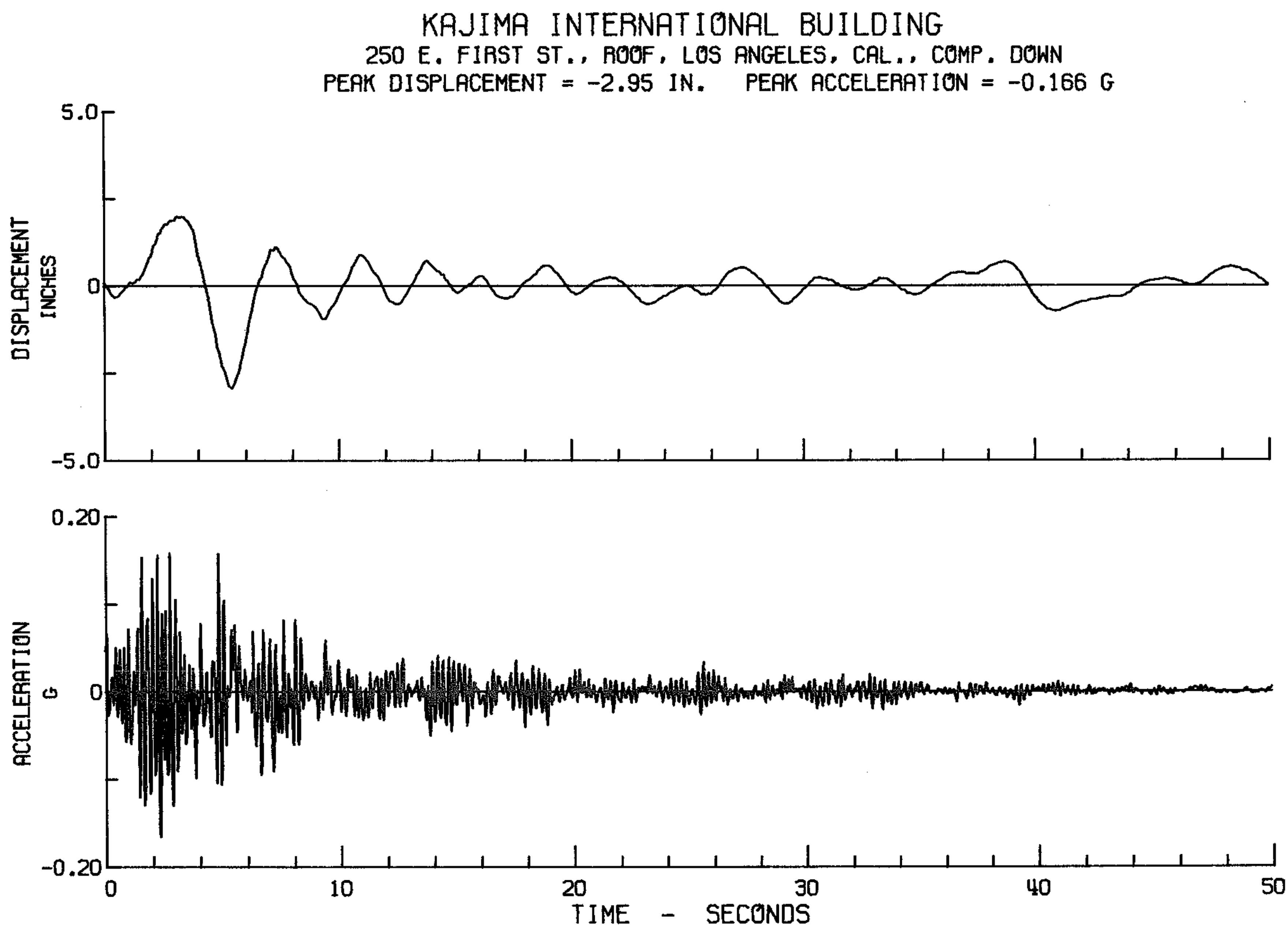

KAJIMA INTERNATIONAL BUILDING
250 E. FIRST ST., ROOF, LOS ANGELES, CAL., COMP. DOWN
PEAK DISPLACEMENT = -2.95 IN.   PEAK ACCELERATION = -0.166 G
DISPLACEMENT
INCHES
5.0
0
-5.0
ACCELERATION
G
0.20
0
-0.20
0
10
20
30
40
50
TIME - SECONDS

Figure 2.15

RESPONSE SPECTRUM

KAJIMA INTERNATIONAL BUILDING

250 E. FIRST ST., BASEMENT, LOS ANGELES, CAL., COMP. DOWN

DAMPING VALUES ARE 0, 2, 5, 10 AND 20 PERCENT OF CRITICAL

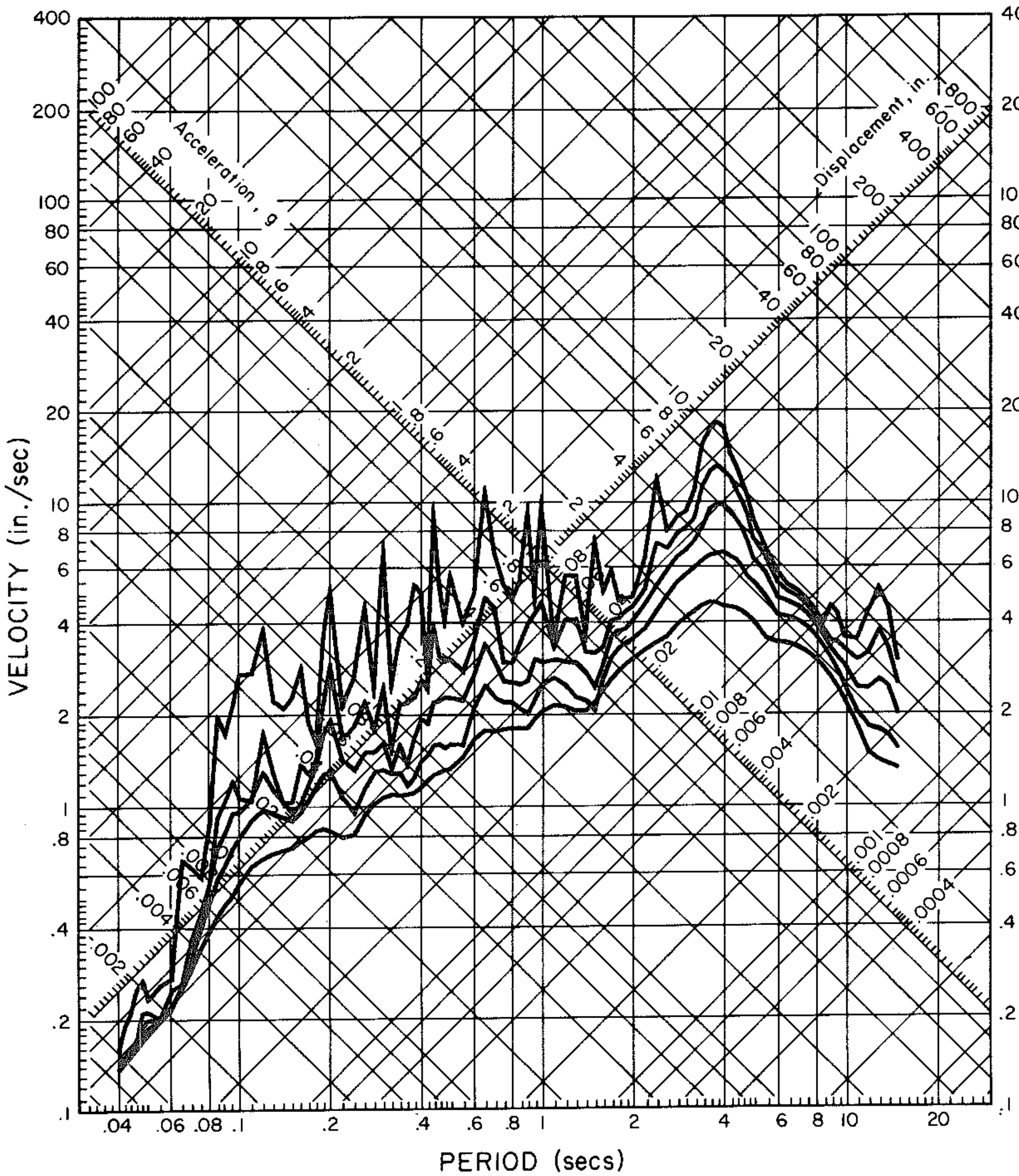


Figure 2.16

Chapter 3

Bank of California Building
15250 Ventura Blvd.
Los Angeles, California

The Bank of California building is a 12-story reinforced concrete frame building located at 15250 Ventura Boulevard approximately 14 miles from the center of the San Fernando earthquake. The building measures approximately 96 by 157 feet in plan and rises 174 feet above grade. This height includes a penthouse which occupies roughly 30 percent of the roof area. The majority of the building is used for office space while the penthouse contains mechanical equipment. The building experienced considerable damage during the earthquake, with approximately 25 percent of the repair costs expended on the epoxy repair of structural members. Figure 3.1 is a picture of the northwest elevation of the Bank of California building.

The lateral load resisting system is composed of several components. Much of the resistance is provided by reinforced concrete moment resisting perimeter frames that extend the full height of the structure and are composed of columns and spandrel beams. Interior frames to the third floor were also designed as moment resisting. Above the third floor, the interior columns and joists were designed to resist only vertical loads. Two shear walls two stories high also provide lateral resistance in the longitudinal direction. An unusual architectural requirement called for the perimeter spandrel beams to be set back approximately 3 feet from the exterior columns at the second floor level. Extensive cracking and minor spalling occurred in the girder stubs that connected the columns to the beams. The foundation system consisted of drilled cast-in-place concrete piles ranging from 35 to 50 feet long. Figure 3.2a is a typical transverse section of the structural system and Figure 3.2b is a typical floor plan of floors 3 through 12.

Strong-motion instruments were placed on the first and seventh floors and on the roof. The three components of recorded acceleration at each level as well as the corresponding integrated displacement are shown in the following figures. Relative displacements were computed and plotted for the horizontal components of motion of the seventh floor and the roof. The response spectra of the recorded ground accelerations in each of the three directions are also provided.

The seismograms show that the Bank of California building was strongly exicted by long period displacements in the ground motion beginning at approximately 10 seconds. This ground motion had a cumulative effect so that the largest displacements of the building occurred 20 to 30 seconds after the earthquake started. These large displacements indicate that the fundamental period of the building in the S79°W direction was about 3.0 seconds during the earthquake. Small amplitude ambient tests conducted after the earthquake produced a fundamental period of 1.6 second. This behavior was the result of both structural and nonstructural damage. The seventh floor and the roof experienced strong high frequency accelerations during the first 10 seconds or so of the earthquake, the result of the higher modes being excited, but these motions did not produce large displacements.

Reference

1. John A. Blume and Associates, "Bank of California", San Fernando, California, Earthquake of February 9, 1971, Leonard M. Murphy (Ed.), U.S. Department of Commerce, NOAA, Washington, D.C., 1973, pp. 327 - 358.

Figure 3.1 Bank of California

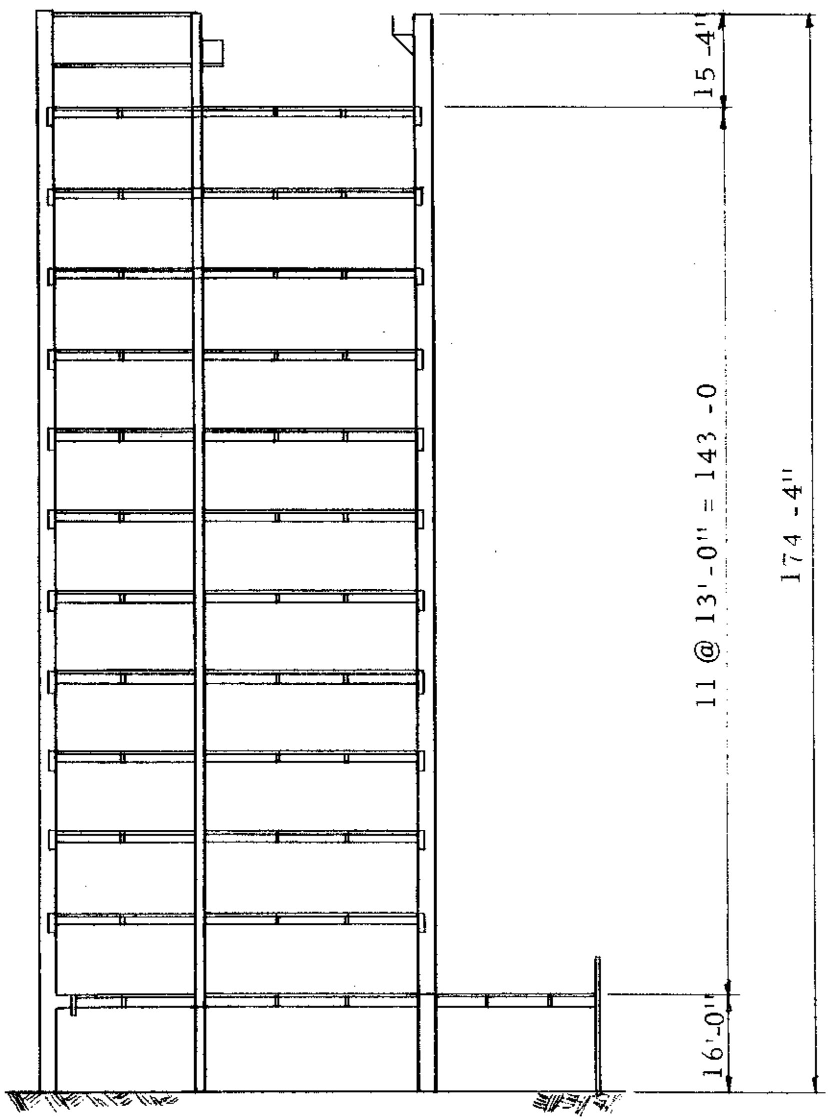


Figure 3.2a Transverse Section

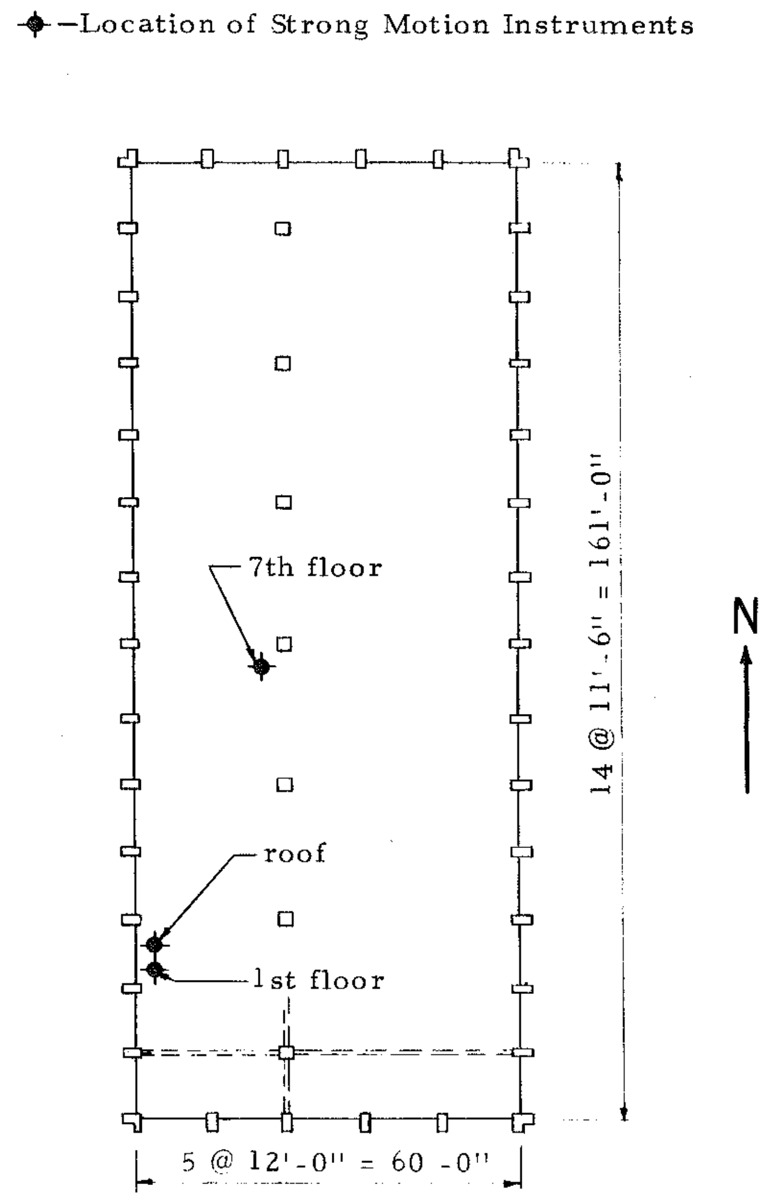
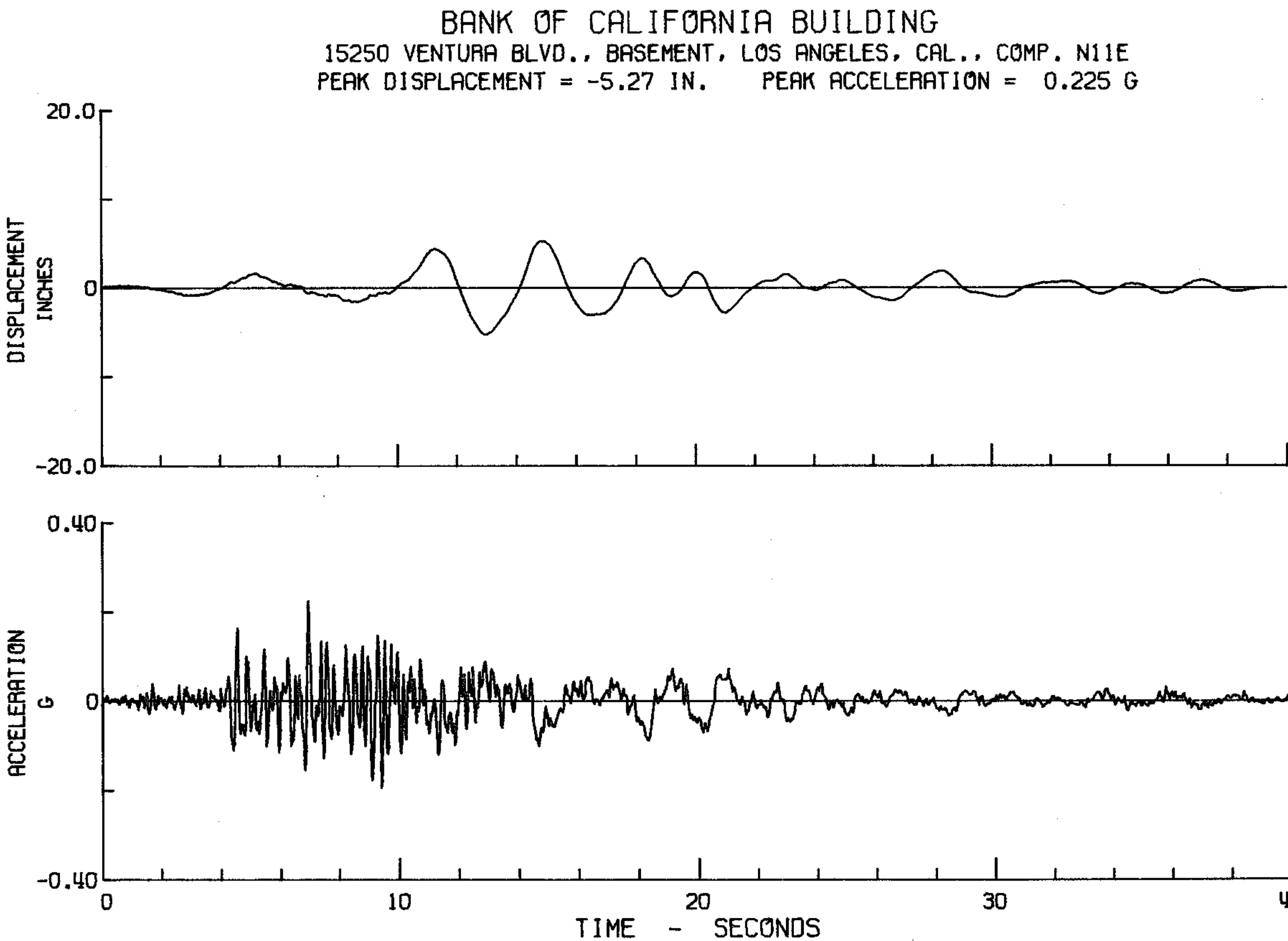


Figure 3.2b Typical Floor Plan

Figure 3.2 Schematic of Bank of California Structural System



BANK OF CALIFORNIA BUILDING
15250 VENTURA BLVD., BASEMENT, LOS ANGELES, CAL., COMP. N11E
PEAK DISPLACEMENT = -5.27 IN.
PEAK ACCELERATION = 0.225 G
DISPLACEMENT
INCHES
20.0
0
-20.0
ACCELERATION
G
0.40
0
-0.40
0
10
20
30
40
TIME - SECONDS

Figure 3.3

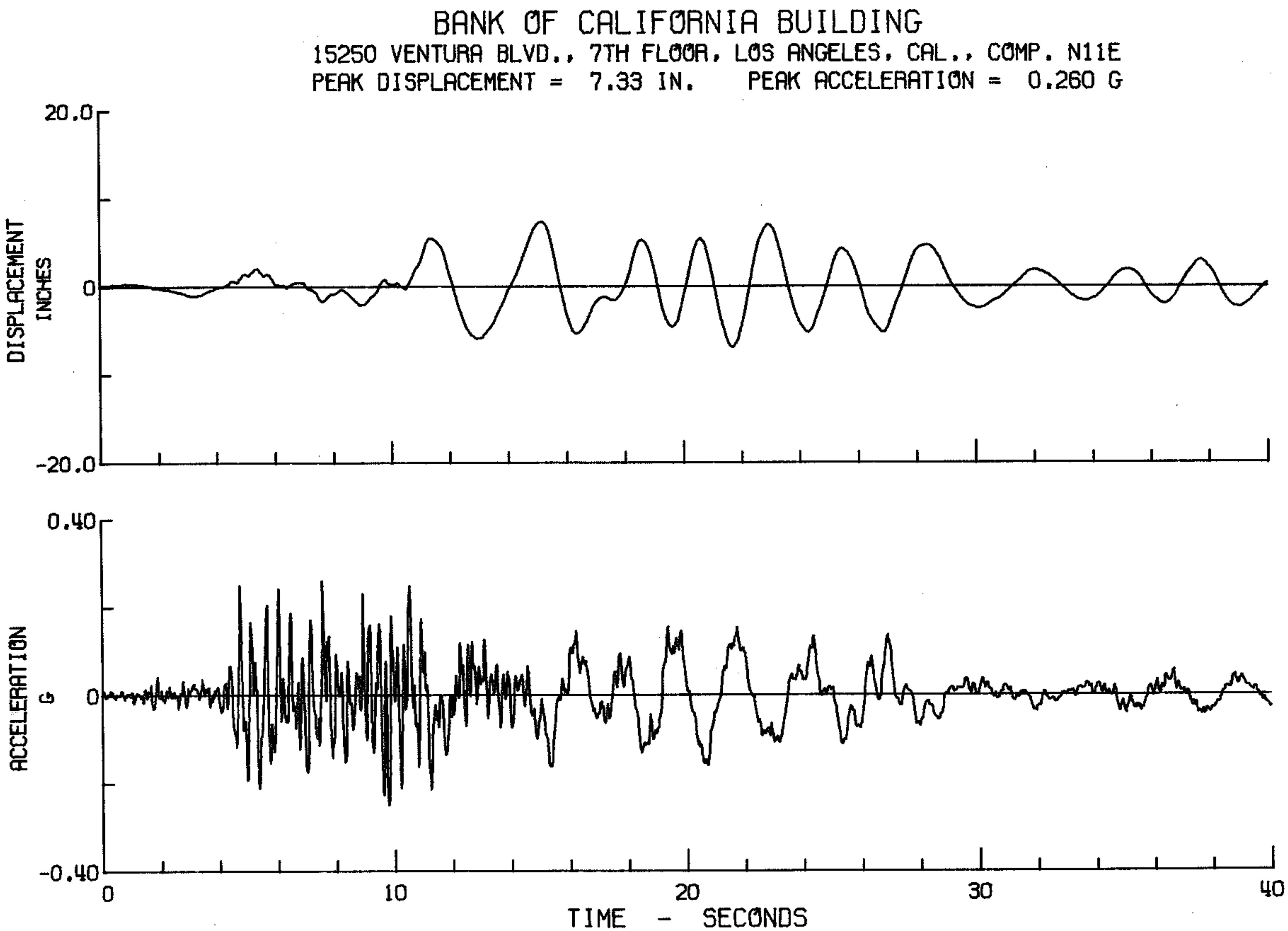

BANK OF CALIFORNIA BUILDING
15250 VENTURA BLVD., 7TH FLOOR, LOS ANGELES, CAL., COMP. N11E
PEAK DISPLACEMENT = 7.33 IN. PEAK ACCELERATION = 0.260 G
20.0
0
-20.0
DISPLACEMENT
INCHES
0.40
0
-0.40
ACCELERATION
G
0
10
20
30
40
TIME - SECONDS

Figure 3.4

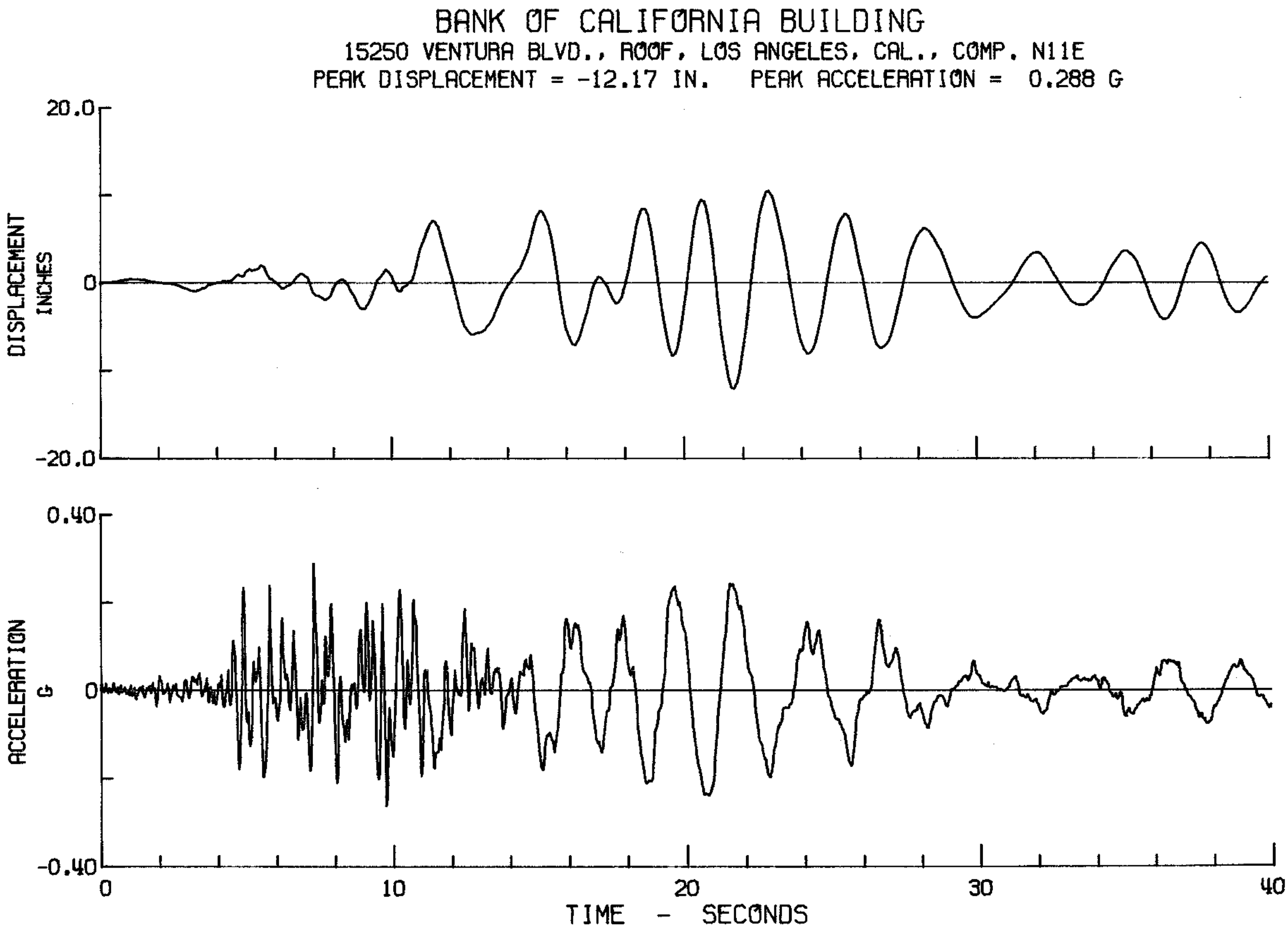

BANK OF CALIFORNIA BUILDING
15250 VENTURA BLVD., ROOF, LOS ANGELES, CAL., COMP. N11E
PEAK DISPLACEMENT = -12.17 IN.   PEAK ACCELERATION = 0.288 G
DISPLACEMENT
INCHES
20.0
0
-20.0
ACCELERATION
G
0.40
0
-0.40
0
10
20
30
40
TIME - SECONDS

Figure 3.5

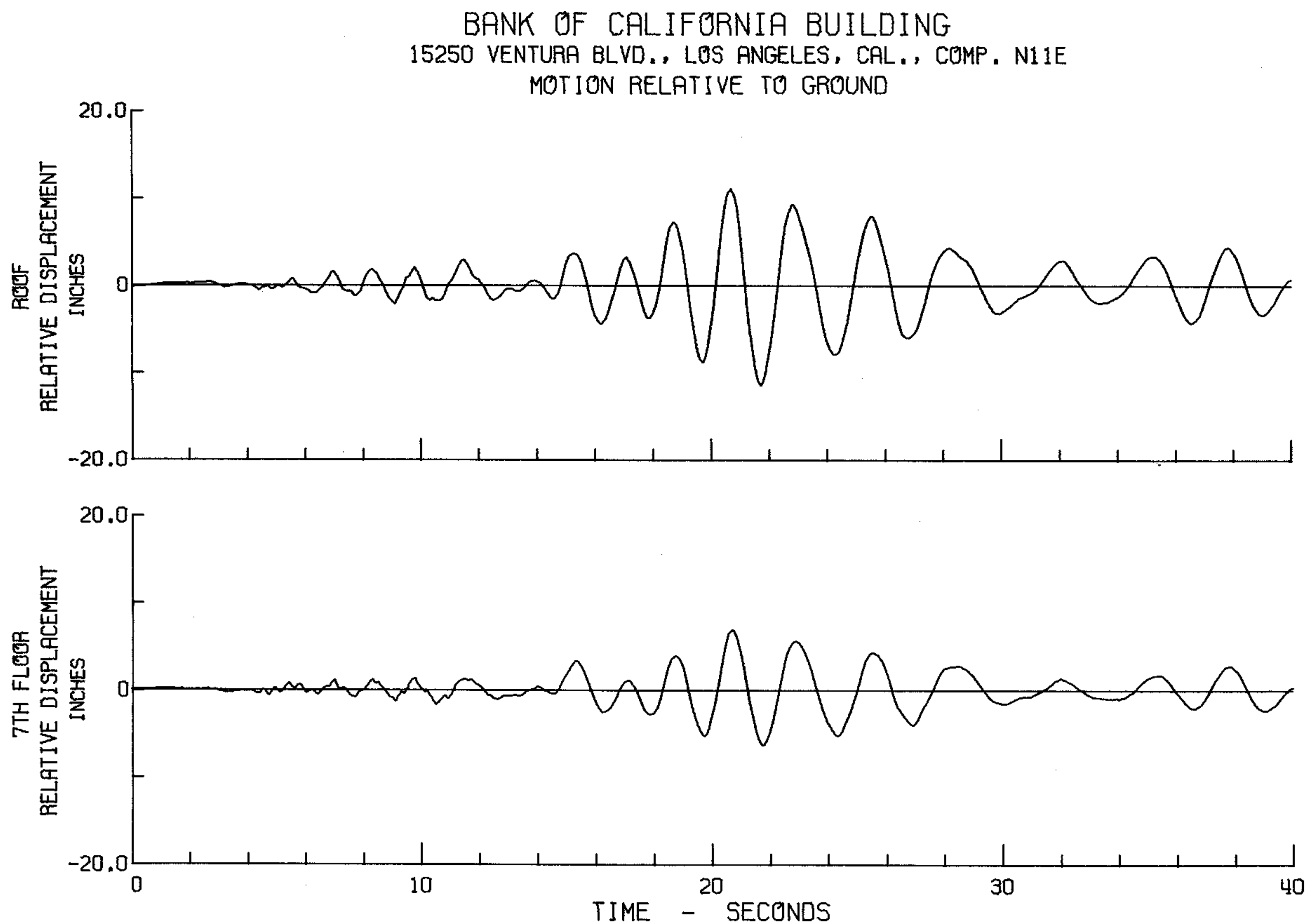

BANK OF CALIFORNIA BUILDING
15250 VENTURA BLVD., LOS ANGELES, CAL., COMP. N11E
MOTION RELATIVE TO GROUND
ROOF
RELATIVE DISPLACEMENT
INCHES
20.0
0
-20.0
7TH FLOOR
RELATIVE DISPLACEMENT
INCHES
20.0
0
-20.0
0
10
20
30
40
TIME - SECONDS

Figure 3.6

RESPONSE SPECTRUM

BANK OF CALIFORNIA BUILDING

15250 VENTURA BLVD., BASEMENT, LOS ANGELES, CAL., COMP. N11E

DAMPING VALUES ARE 0, 2, 5, 10 AND 20 PERCENT OF CRITICAL

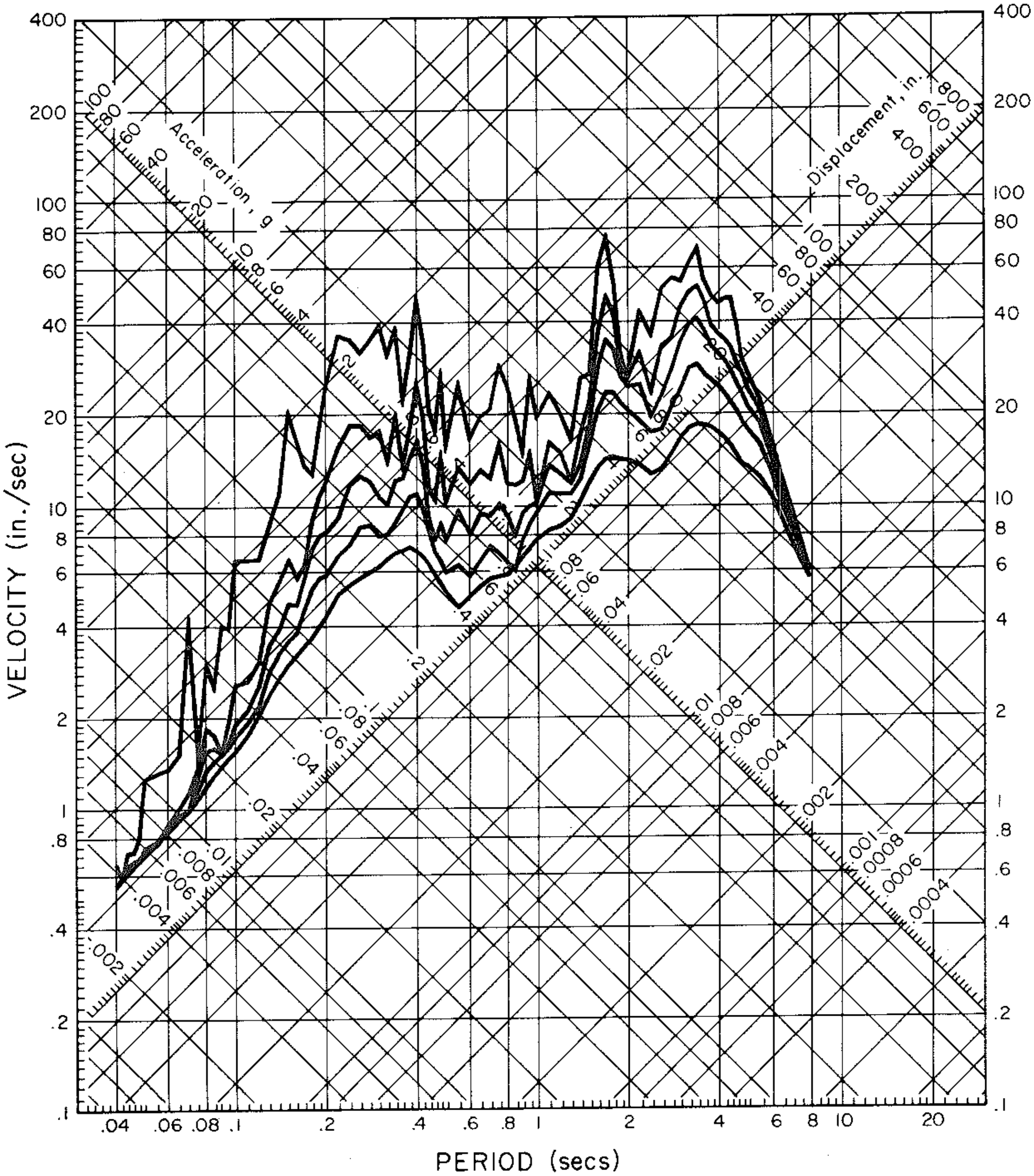


Figure 3.7

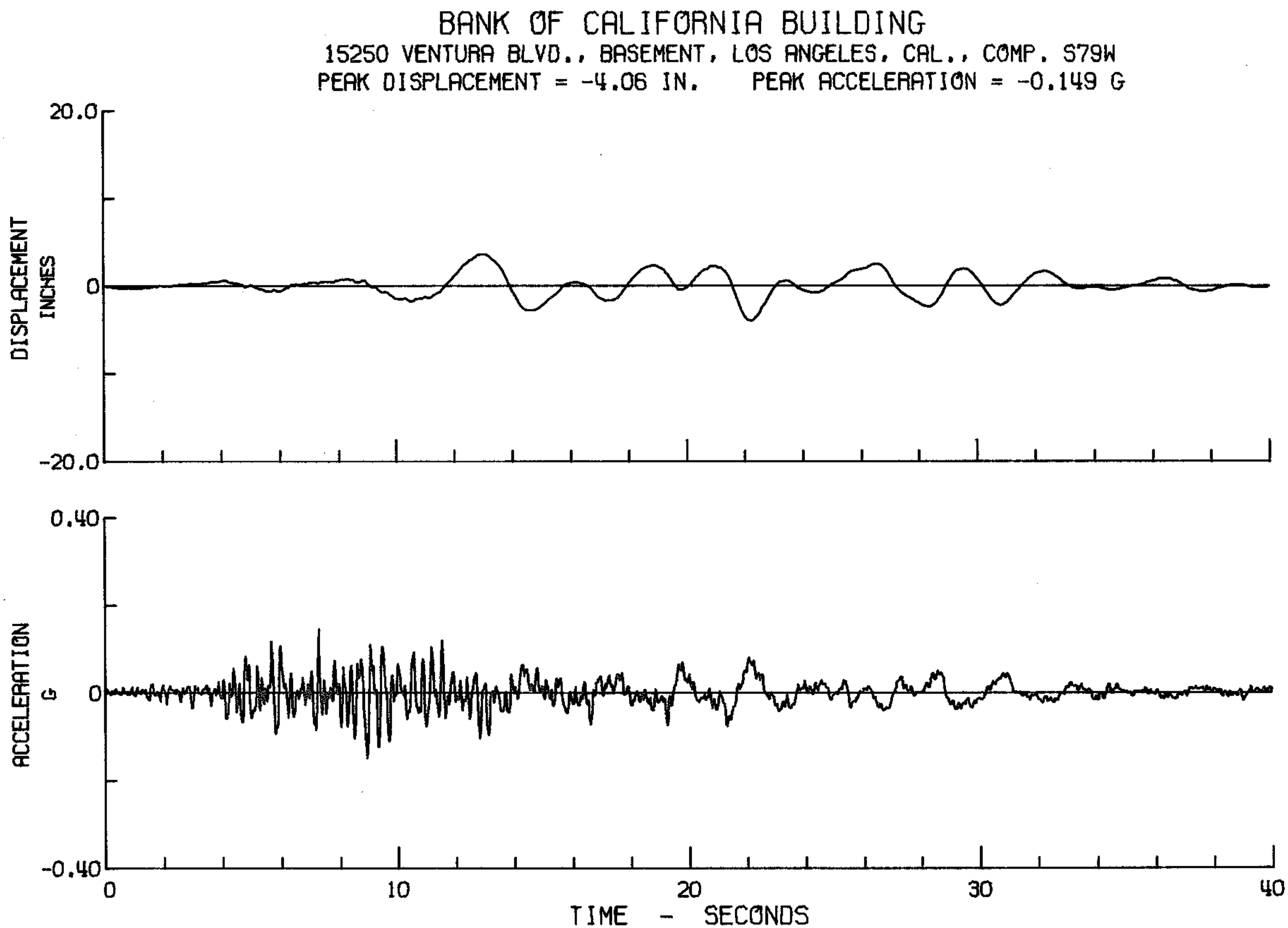

BANK OF CALIFORNIA BUILDING
15250 VENTURA BLVD., BASEMENT, LOS ANGELES, CAL., COMP. S79W
PEAK DISPLACEMENT = -4.06 IN.
PEAK ACCELERATION = -0.149 G
20.0
0
-20.0
DISPLACEMENT
INCHES
0.40
0
-0.40
ACCELERATION
G
0
10
20
30
40
TIME - SECONDS

Figure 3.8

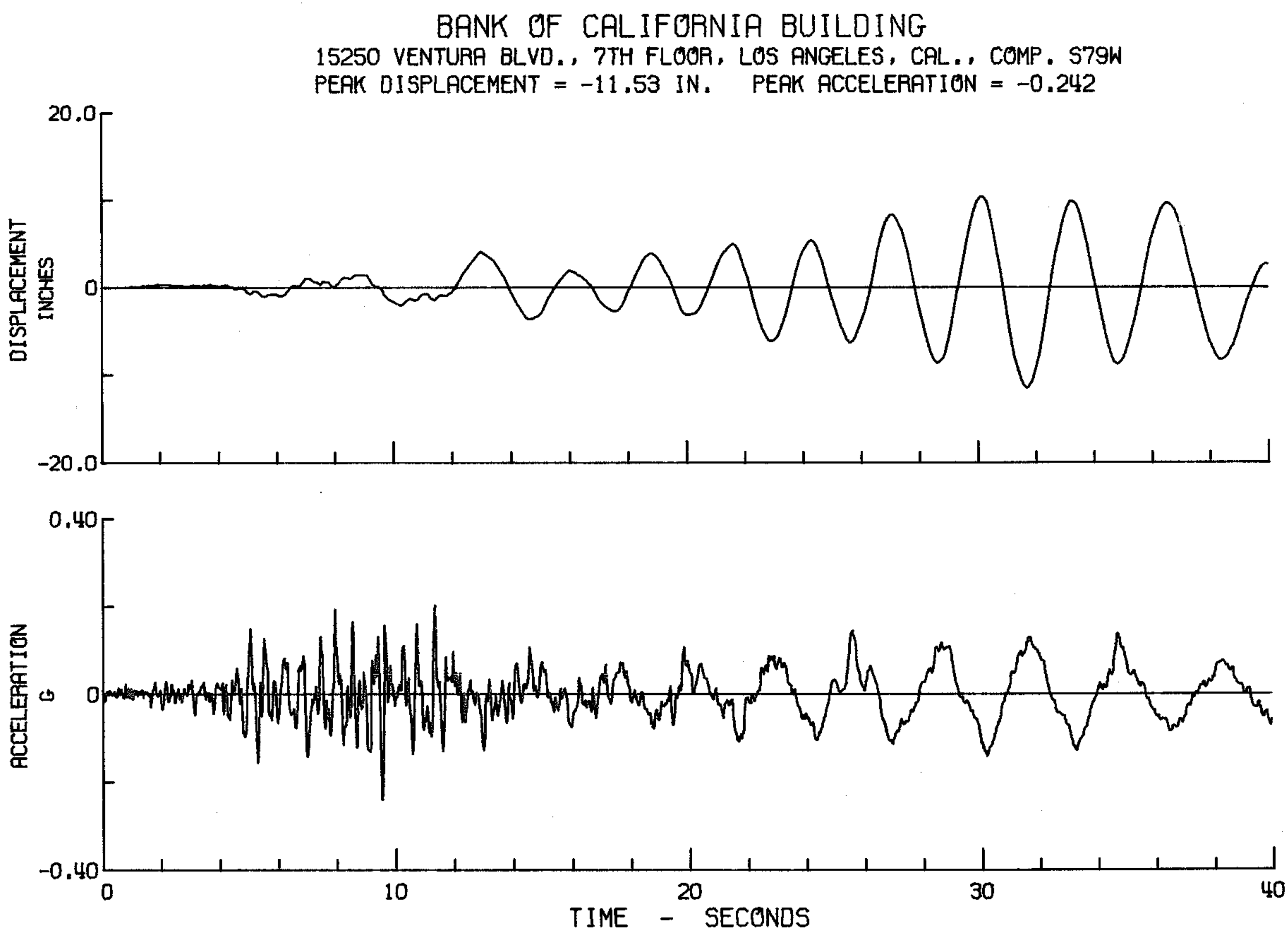

BANK OF CALIFORNIA BUILDING
15250 VENTURA BLVD., 7TH FLOOR, LOS ANGELES, CAL., COMP. S79W
PEAK DISPLACEMENT = -11.53 IN.
PEAK ACCELERATION = -0.242
DISPLACEMENT
INCHES
20.0
0
-20.0
ACCELERATION
G
0.40
0
-0.40
0
10
20
30
40
TIME - SECONDS

Figure 3.9

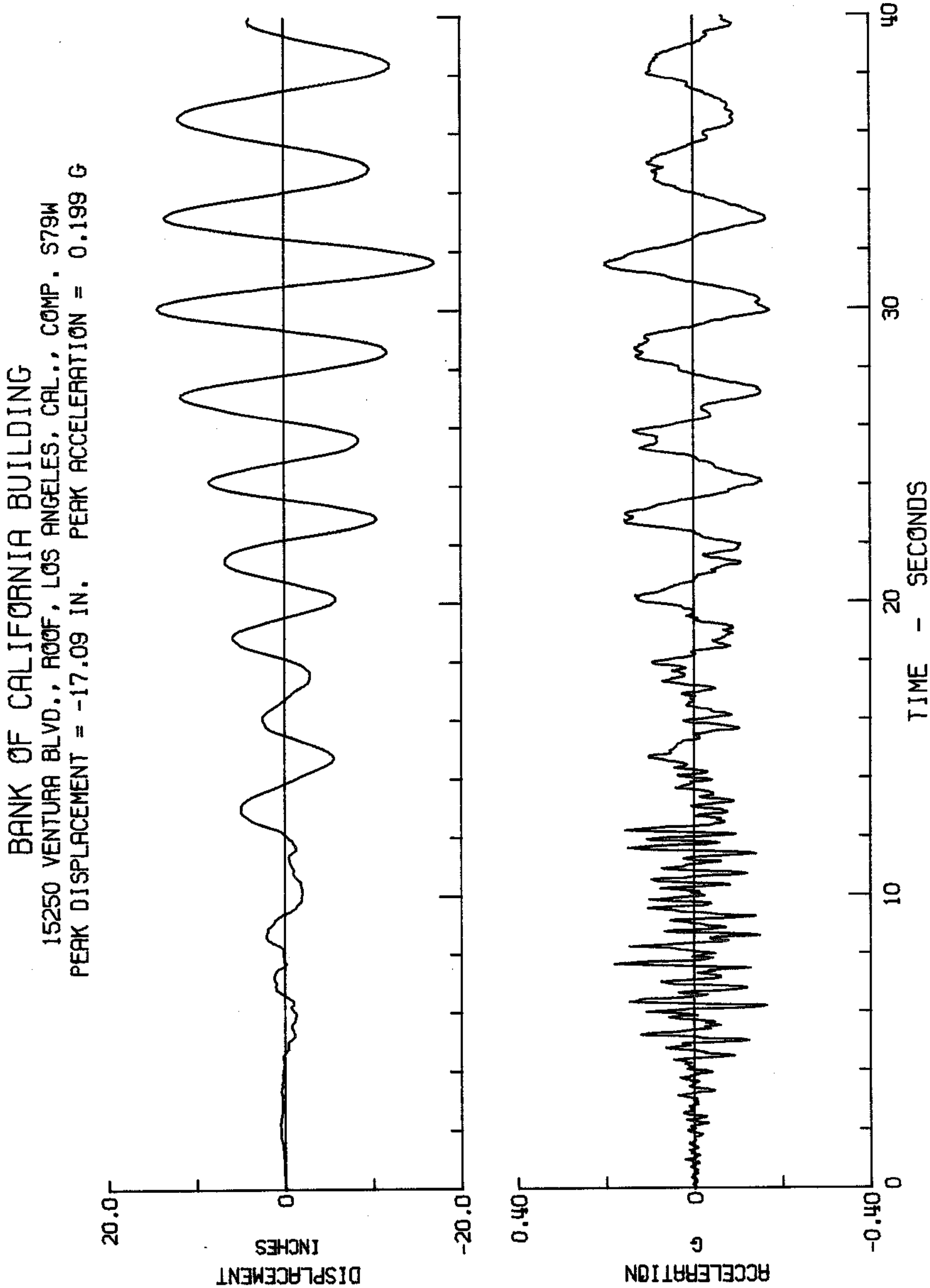

BANK OF CALIFORNIA BUILDING
15250 VENTURA BLVD., ROOF, LOS ANGELES, CAL., COMP. S79W
PEAK DISPLACEMENT = -17.09 IN.   PEAK ACCELERATION = 0.199 G
DISPLACEMENT
INCHES
20.0
0
-20.0
ACCELERATION
G
0.40
0
-0.40
0
10
20
30
40
TIME - SECONDS

Figure 3.10

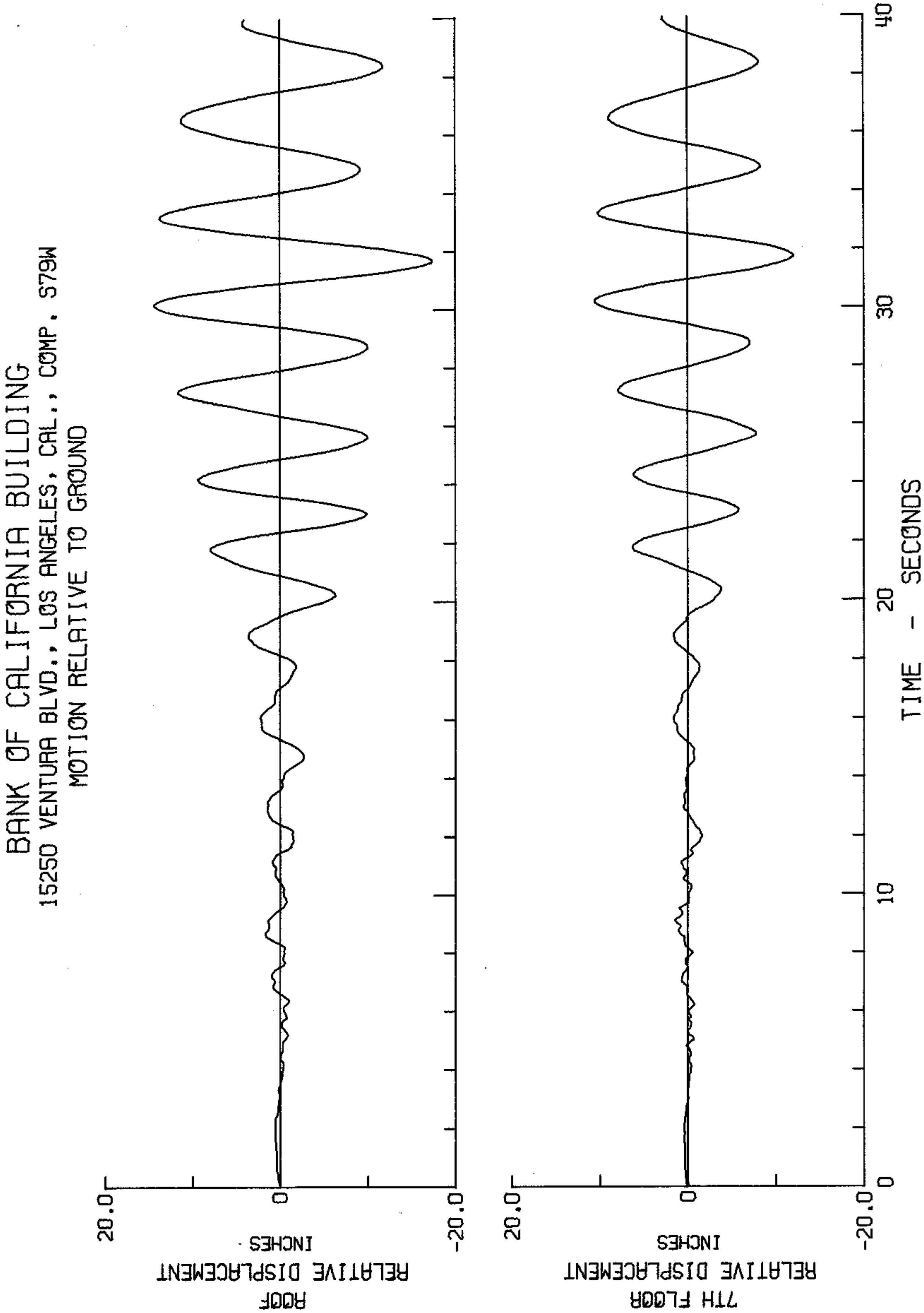

BANK OF CALIFORNIA BUILDING
15250 VENTURA BLVD., LOS ANGELES, CAL., COMP. S79W
MOTION RELATIVE TO GROUND
ROOF
RELATIVE DISPLACEMENT
INCHES
20.0
0
-20.0
7TH FLOOR
RELATIVE DISPLACEMENT
INCHES
20.0
0
-20.0
0
10
20
30
40
TIME - SECONDS

Figure 3.11

RESPONSE SPECTRUM

BANK OF CALIFORNIA BUILDING

15250 VENTURA BLVD., BASEMENT, LOS ANGELES, CAL., COMP. S79W

DAMPING VALUES ARE 0, 2, 5, 10 AND 20 PERCENT OF CRITICAL

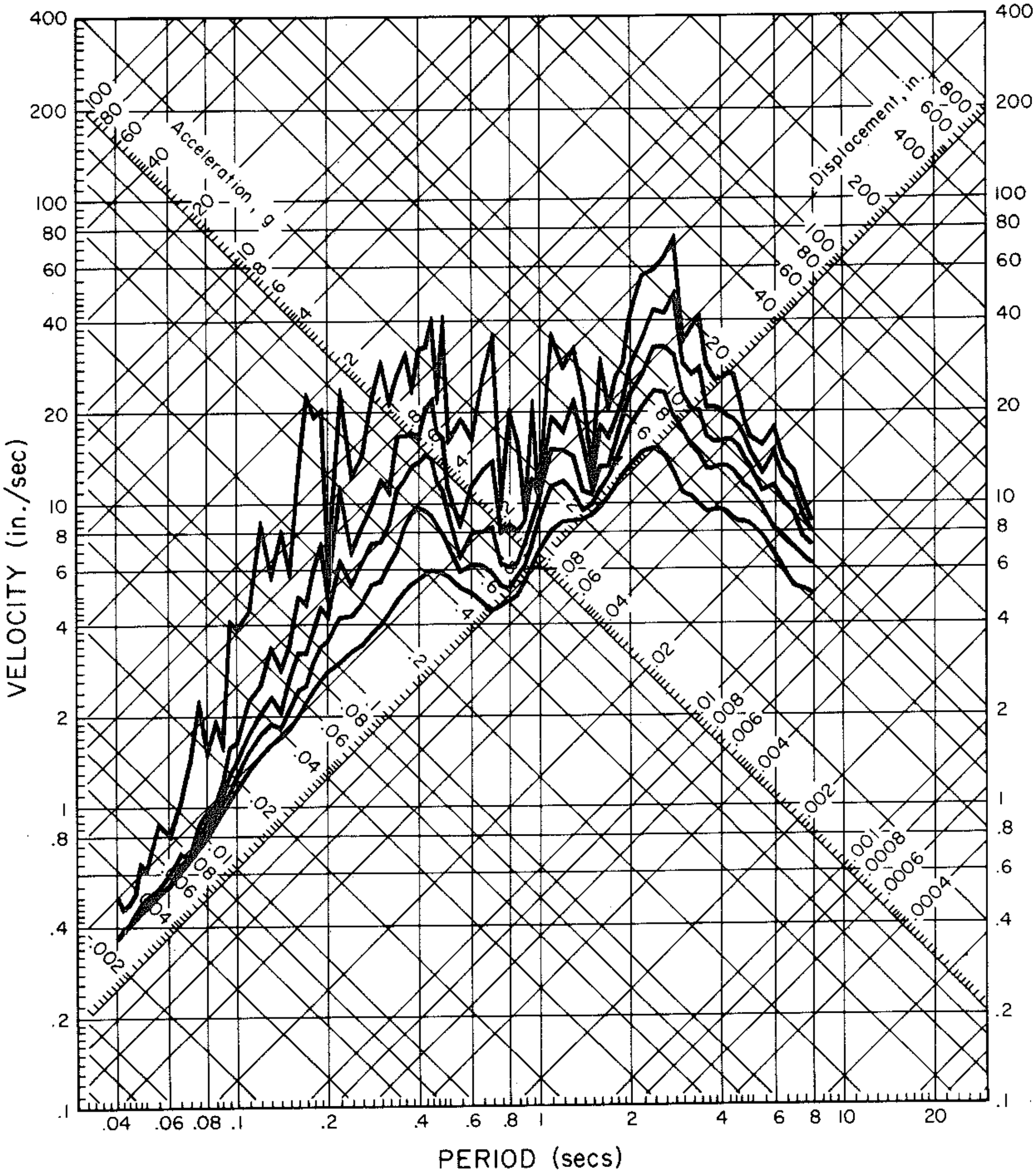


Figure 3.12

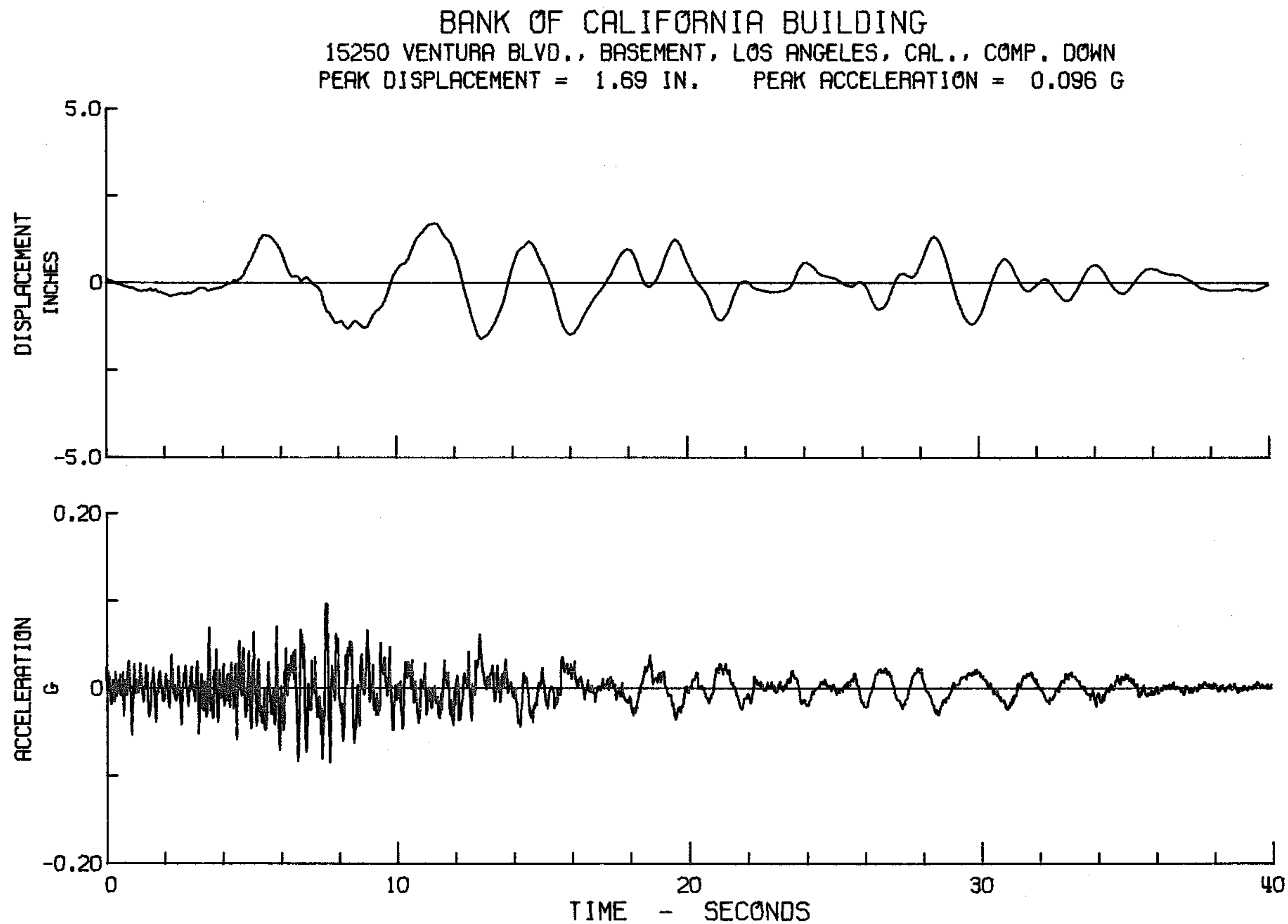

BANK ØF CALIFØRNIA BUILDING
15250 VENTURA BLVD., BASEMENT, LØS ANGELES, CAL., CØMP. DØWN
PEAK DISPLACEMENT = 1.69 IN.   PEAK ACCELERATIØN = 0.096 G
DISPLACEMENT
INCHES
5.0
0
-5.0
ACCELERATIØN
G
0.20
0
-0.20
0
10
20
30
40
TIME - SECØNDS

Figure 3.13

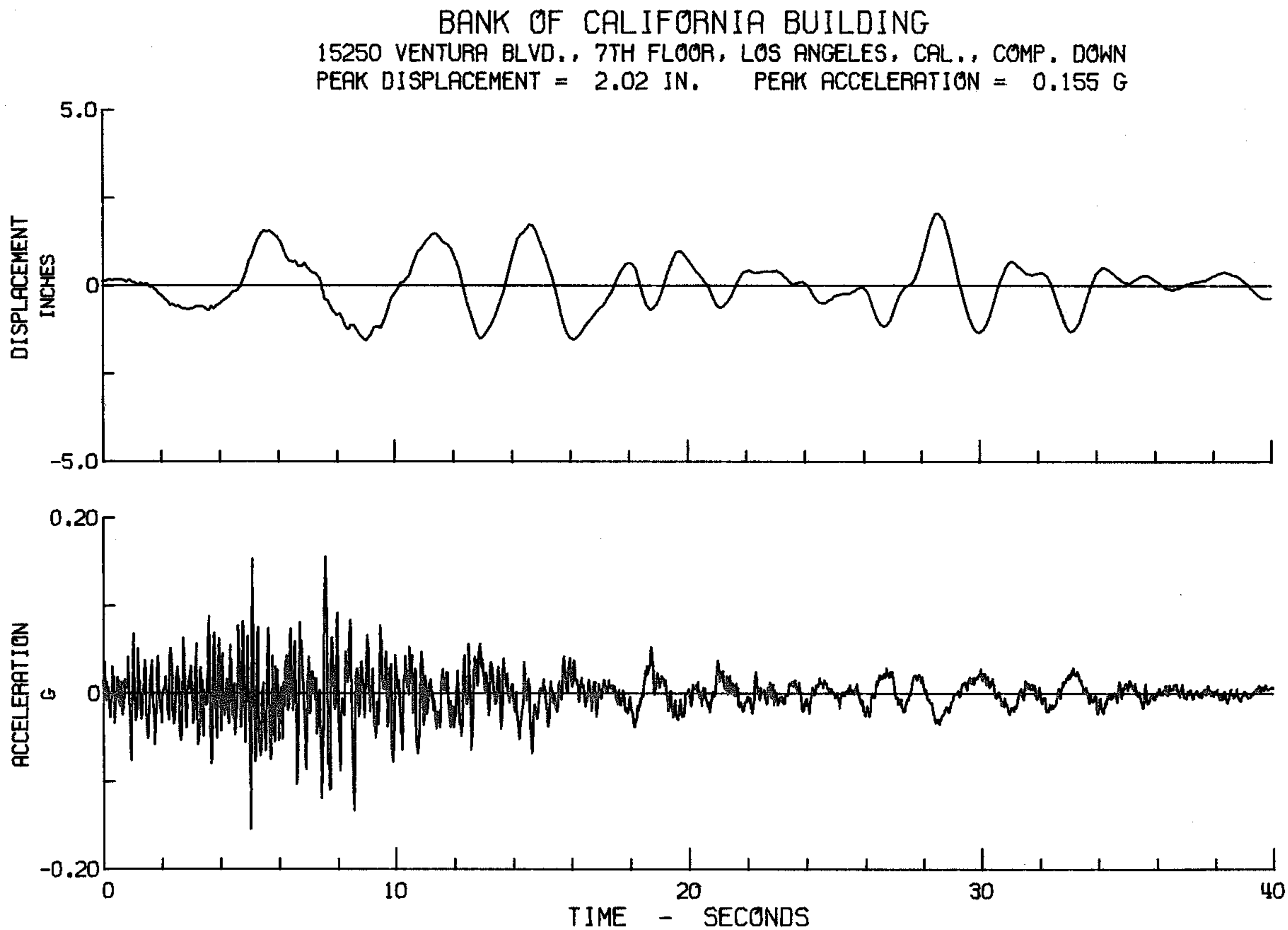

BANK OF CALIFORNIA BUILDING
15250 VENTURA BLVD., 7TH FLOOR, LOS ANGELES, CAL., COMP. DOWN
PEAK DISPLACEMENT = 2.02 IN. PEAK ACCELERATION = 0.155 G
DISPLACEMENT
INCHES
5.0
0
-5.0
ACCELERATION
G
0.20
0
-0.20
0
10
20
30
40
TIME - SECONDS

Figure 3.14

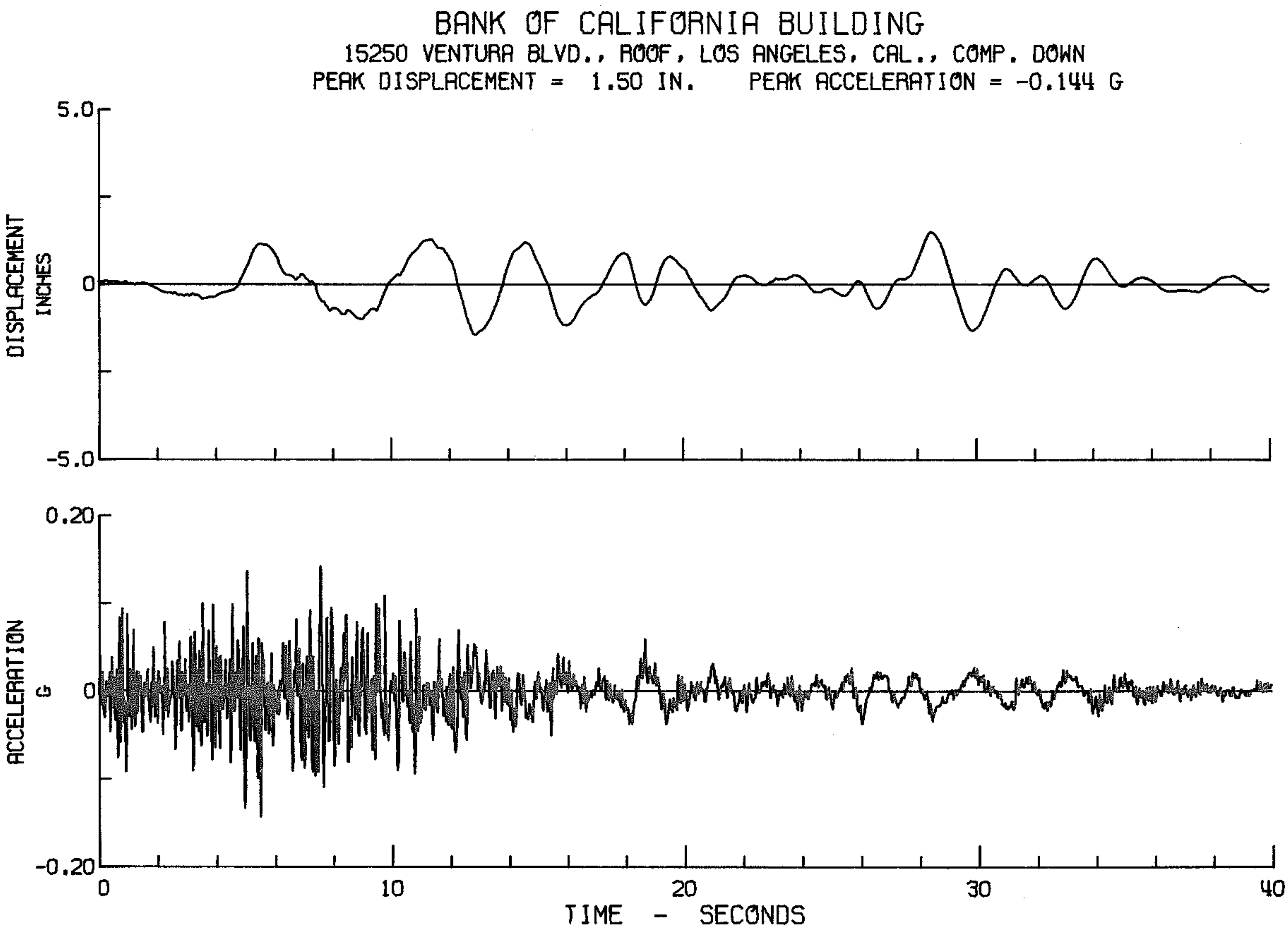

BANK ØF CALIFØRNIA BUILDING
15250 VENTURA BLVD., RØØF, LØS ANGELES, CAL., CØMP. DØWN
PEAK DISPLACEMENT = 1.50 IN. PEAK ACCELERATIØN = -0.144 G
DISPLACEMENT INCHES
5.0
0
-5.0
ACCELERATIØN G
0.20
0
-0.20
0
10
20
30
40
TIME - SECØNDS

Figure 3.15

RESPONSE SPECTRUM

BANK OF CALIFORNIA BUILDING

15250 VENTURA BLVD., BASEMENT, LOS ANGELES, CAL., COMP. DOWN

DAMPING VALUES ARE 0, 2, 5, 10 AND 20 PERCENT OF CRITICAL

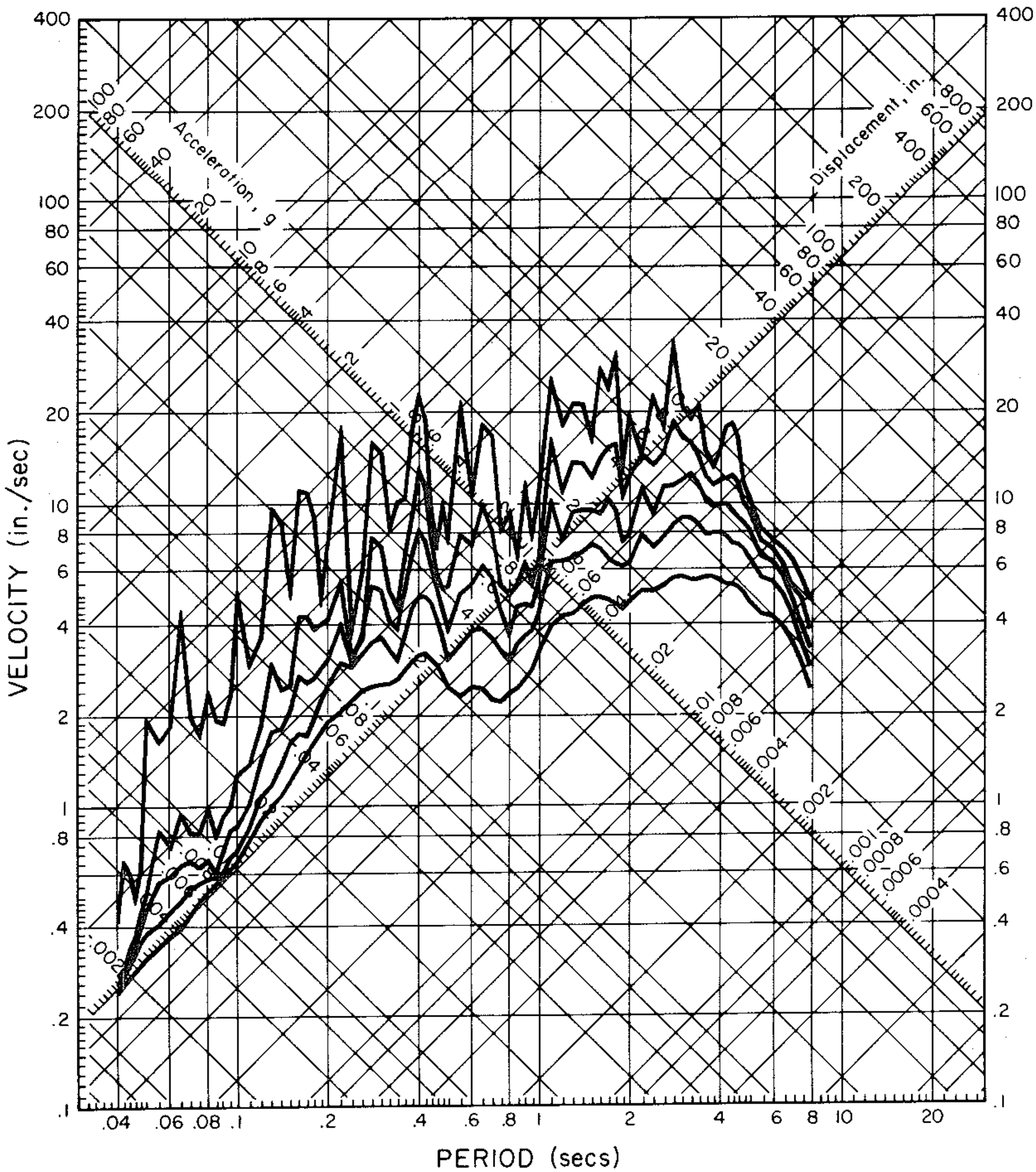


Figure 3.16

Chapter 4

Holiday Inn
8244 Orion Avenue
Los Angeles, California

The Holiday Inn site at 8244 Orion Avenue is approximately 8 miles from the center of energy release of the San Fernando earthquake of February 9, 1971. The seven-story reinforced concrete frame building, designed in 1965, measures 61 by 150 feet in plan and extends 65 feet above grade. The Holiday Inn was the closest instrumented building to the center of the earthquake and it suffered both structural and nonstructural damage amounting to $145,000, which is 11 percent of the initial construction cost. The concrete frame was cracked in many places and every guest room sustained nonstructural damage There was no loss of life to the occupants of the structure which experienced accelerations of nearly 40% g at the roof level. The damaged frame was repaired with epoxy and the building was rehabilitated following the earthquake. Some nonstructural damage was also sustained in a later aftershock. Figure 4. 1 is a picture of the north face of the Holiday Inn.

The structural system of the Holiday Inn is composed of reinforced concrete column and slab construction with deep spandrel beams around the perimeter. Consequently, the exterior frames are approximately twice as stiff as the interior frames. Partitions are constructed of wallboard on metal studs. Capped, poured, reinforced concrete friction piles serve as the foundation system. Figure 4. 2a is a typical transverse section of the structural system of the Holiday Inn and Figure 4. 2b is a typical floor plan.

Strong-motion instruments were located on the first and fourth floors as well as the roof. The instruments were designed to start simultaneously and all three instruments functioned properly during the earthquake. Three components of acceleration were recorded at each level and are presented in the following figures. Also presented in the figures are the integrated displacements for all records as well as the calculated relative displacements in the horizontal direction. Response spectra are also included for the motion recorded at the first floor level. The damage sustained by the building undoubtedly had a strong effect on the dynamic properties of the structure and the resulting nonlinear behavior would affect the recorded motions. The very long period motion in the relative displacements are possibly a spurious result of the digitization and data processing in this instance.

Reference

1. John A. Blume and Associates, "Holiday Inn", San Fernando, California, Earthquake of February 9, 1971, Leonard M. Murphy (Ed.), U. S. Department of Commerce, NOAA, Washington, D.C., 1973, pp. 359 - 394.

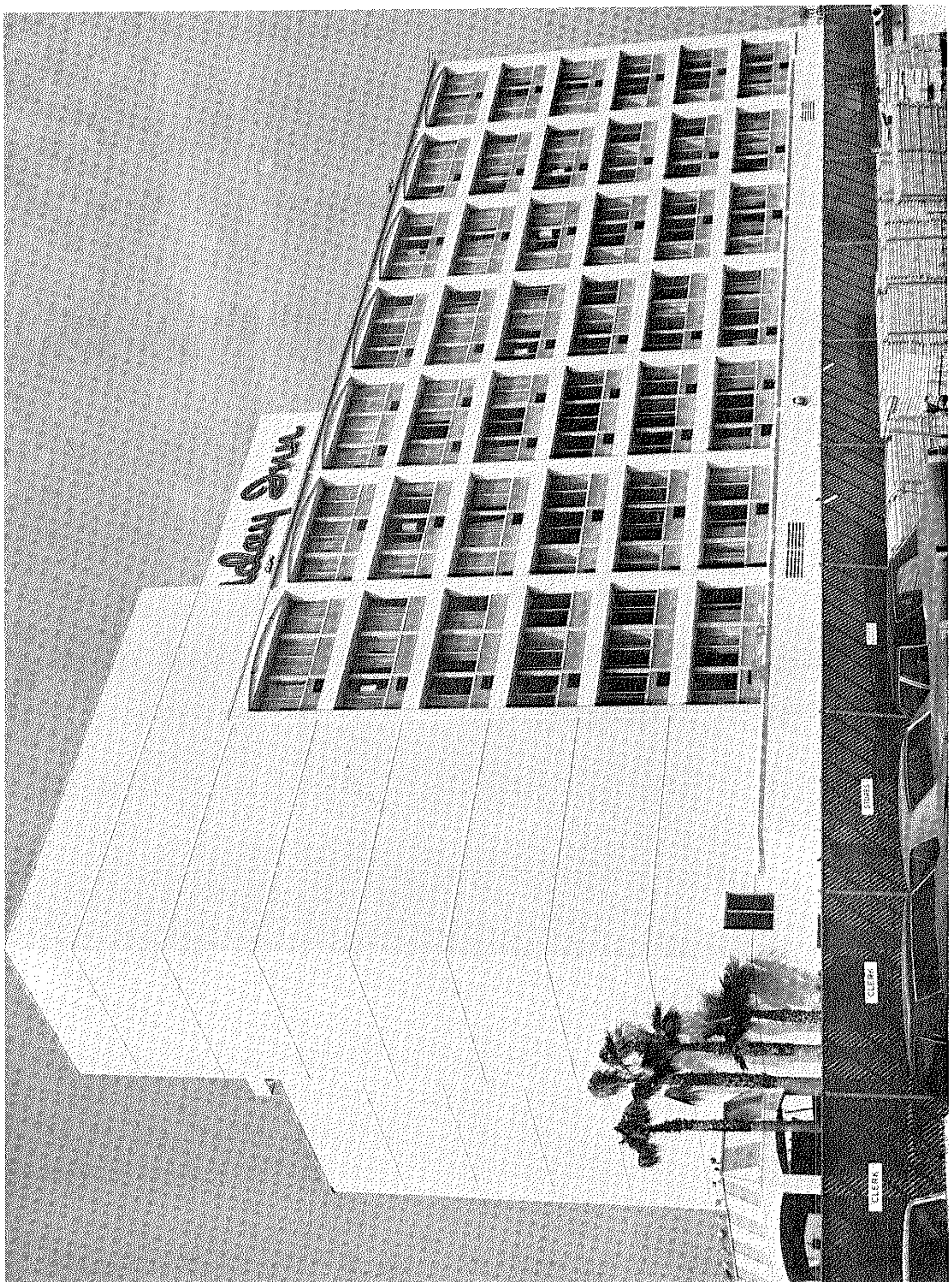


Figure 4.1 Holiday Inn

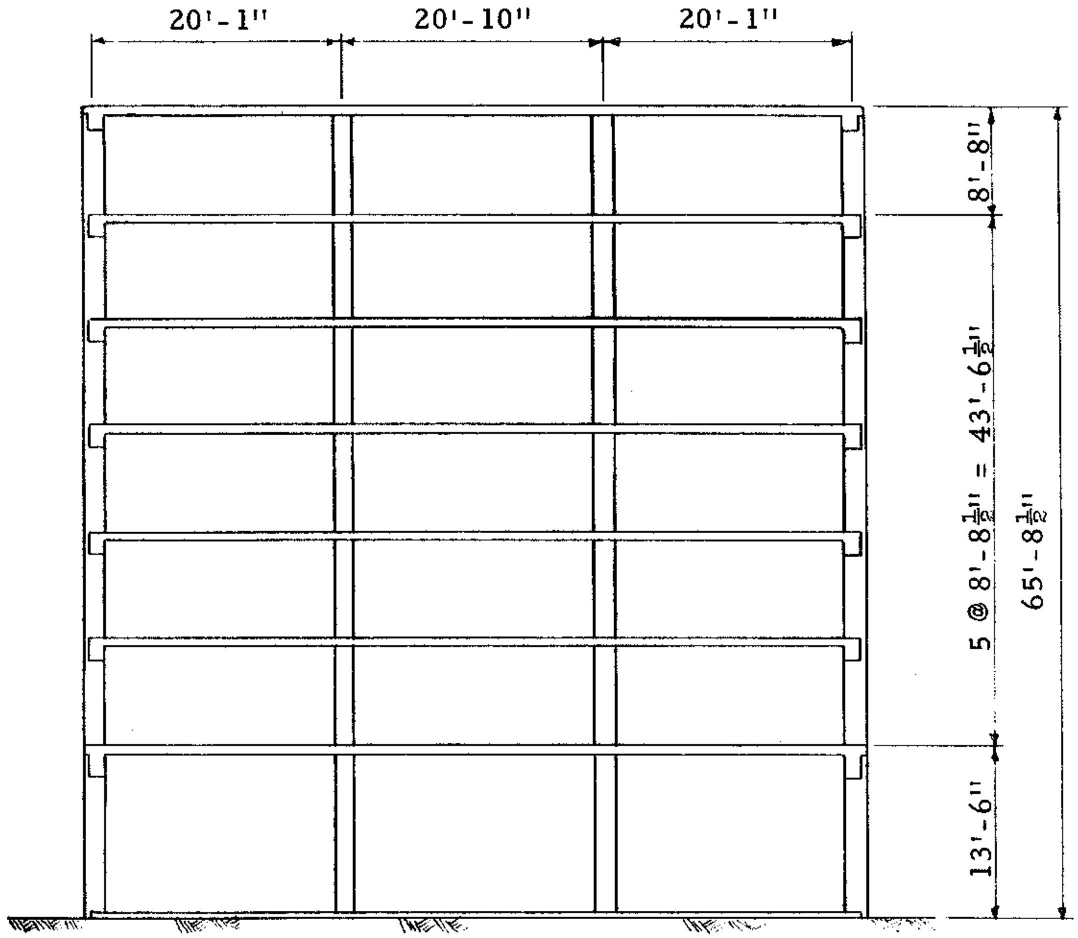


Figure 4.2a Transverse Section

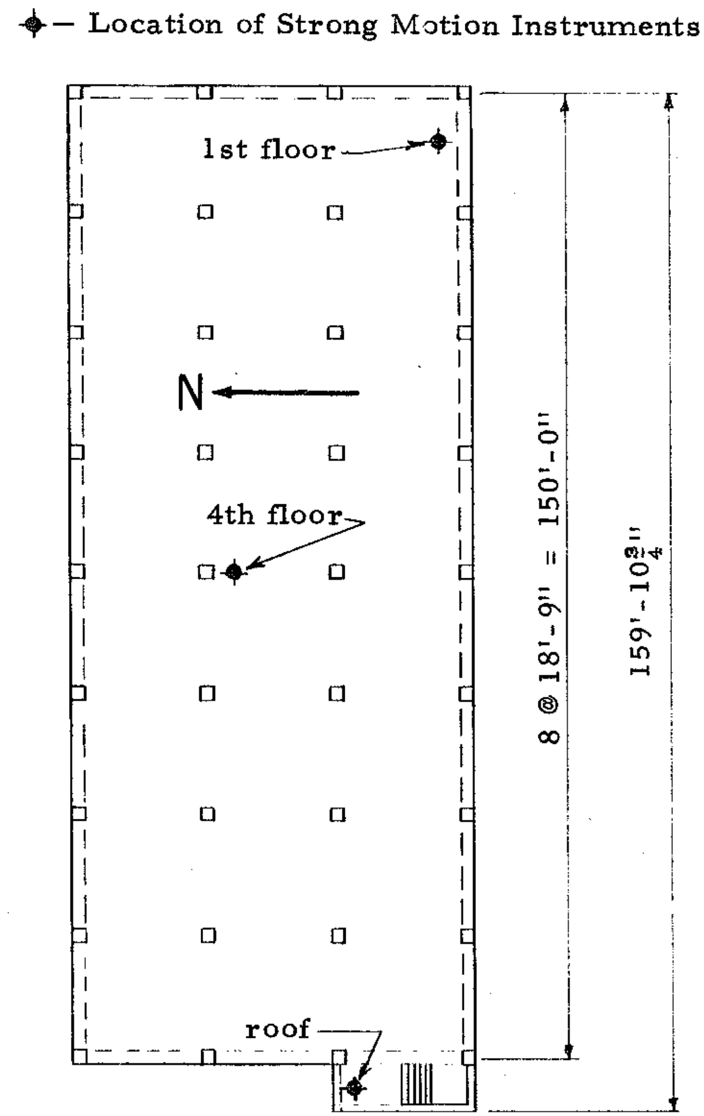


Figure 4.2b Typical Floor Plan

Figure 4.2 Schematic of Holiday Inn Structural System

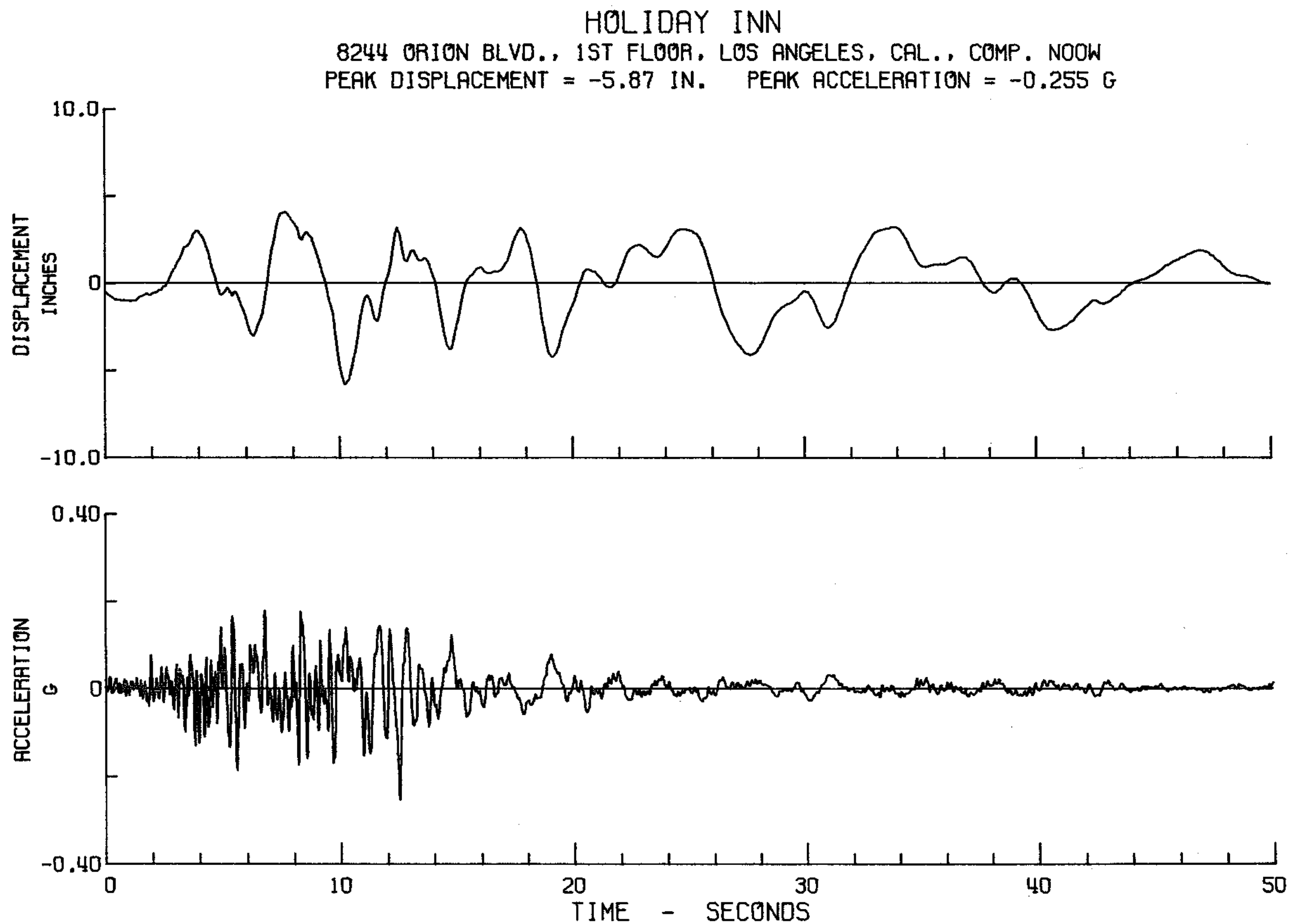

HOLIDAY INN
8244 ORION BLVD., 1ST FLOOR, LOS ANGELES, CAL., COMP. NOOW
PEAK DISPLACEMENT = -5.87 IN.   PEAK ACCELERATION = -0.255 G
10.0
0
-10.0
DISPLACEMENT
INCHES
0.40
0
-0.40
ACCELERATION
G
0
10
20
30
40
50
TIME - SECONDS

Figure 4.3

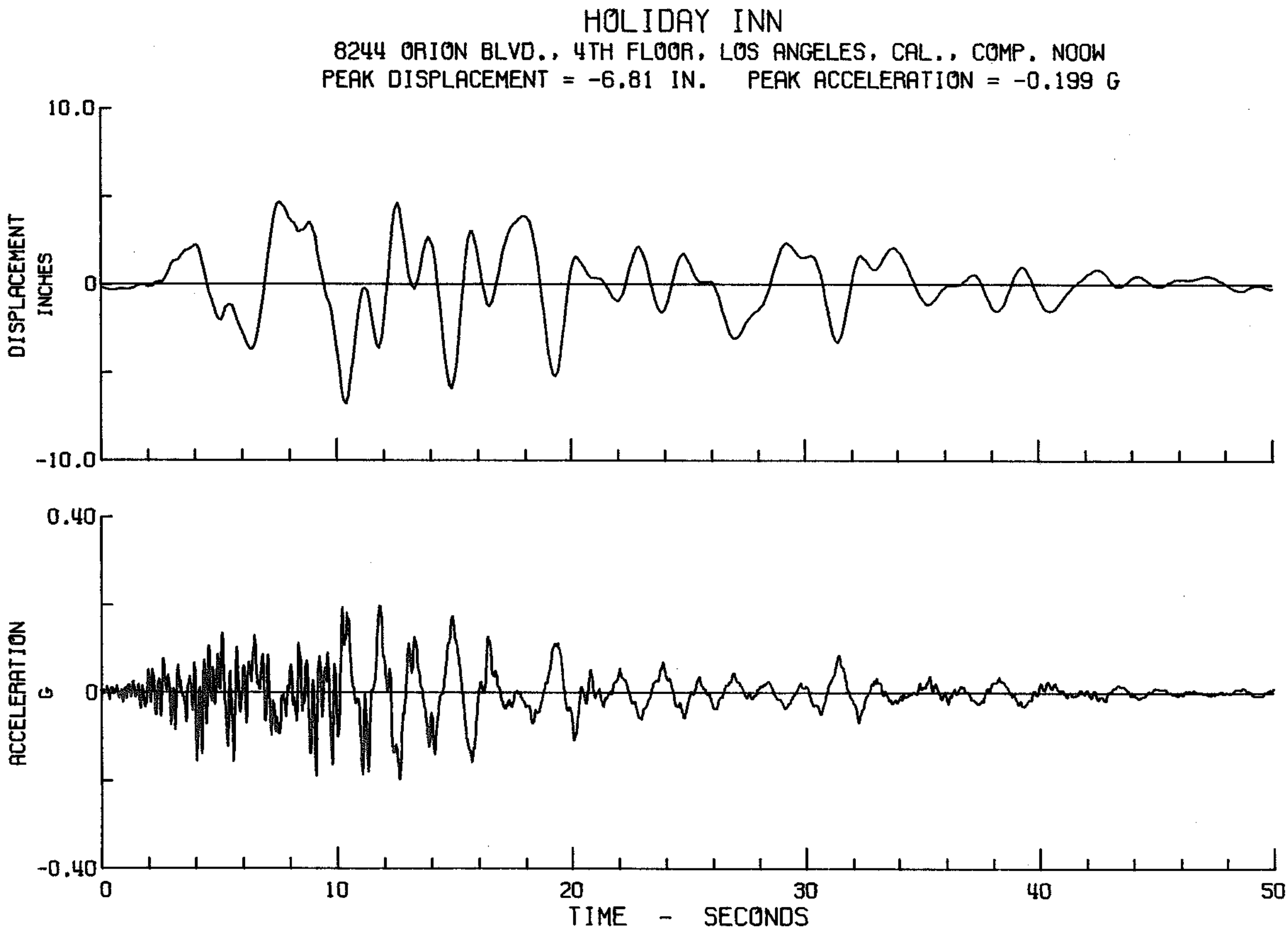

HOLIDAY INN
8244 ORION BLVD., 4TH FLOOR, LOS ANGELES, CAL., COMP. NOOW
PEAK DISPLACEMENT = -6.81 IN.   PEAK ACCELERATION = -0.199 G
10.0
0
-10.0
DISPLACEMENT
INCHES
0.40
0
-0.40
ACCELERATION
G
0
10
20
30
40
50
TIME - SECONDS

Figure 4.4

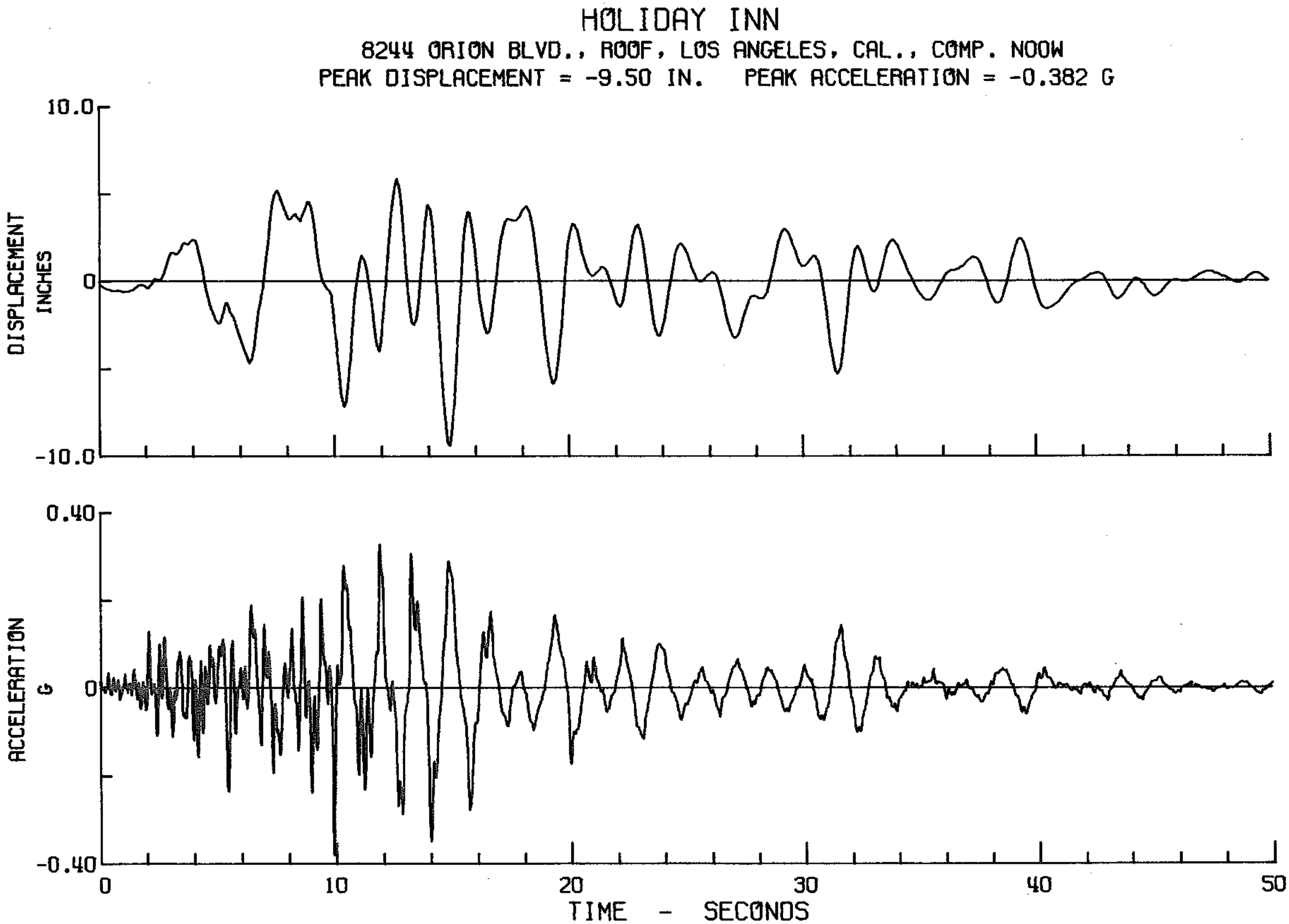

HOLIDAY INN
8244 ORION BLVD., ROOF, LOS ANGELES, CAL., COMP. NOOW
PEAK DISPLACEMENT = -9.50 IN.   PEAK ACCELERATION = -0.382 G
DISPLACEMENT INCHES
10.0
0
-10.0
ACCELERATION G
0.40
0
-0.40
0
10
20
30
40
50
TIME - SECONDS

Figure 4.5

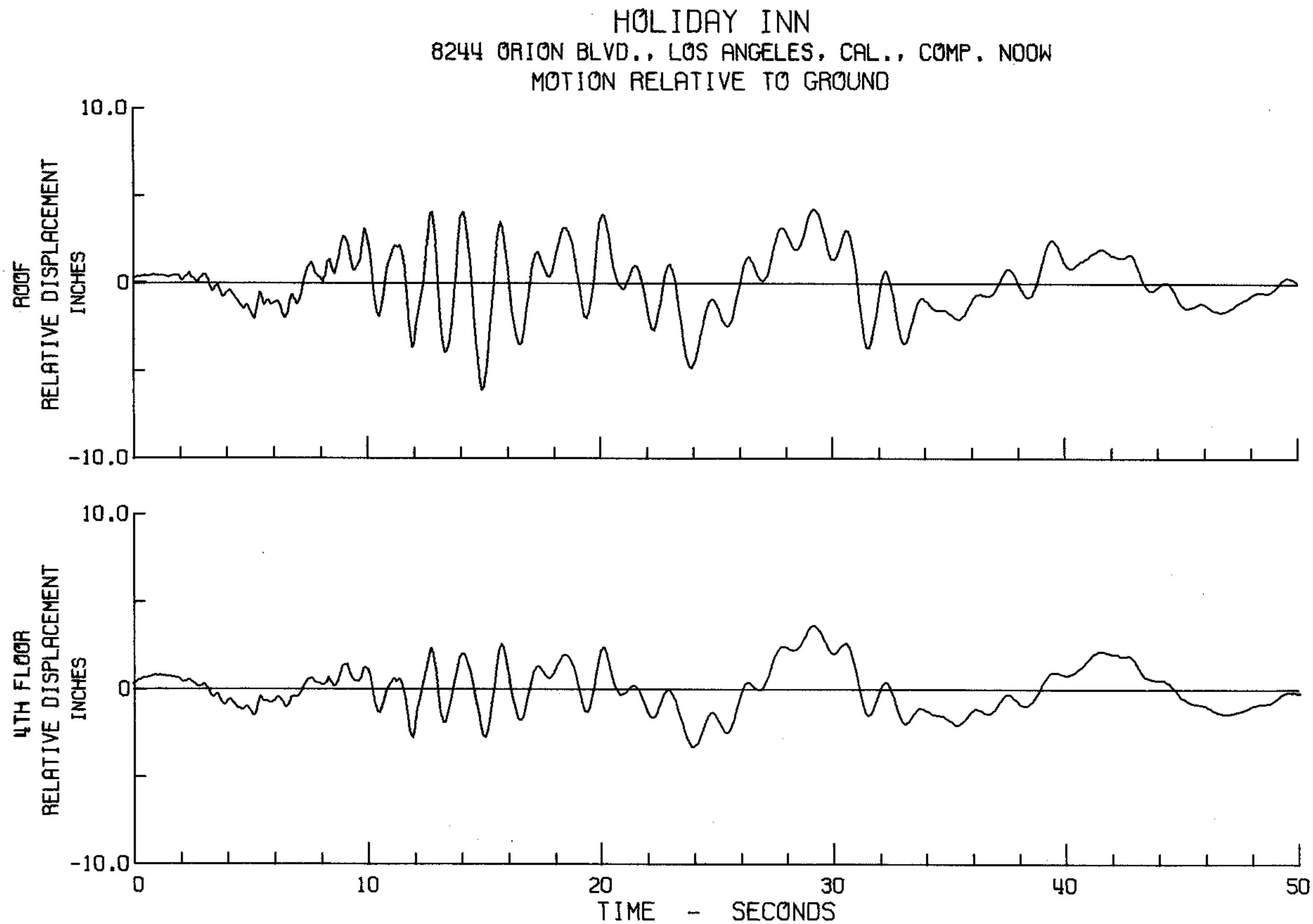

HOLIDAY INN
8244 ORION BLVD., LOS ANGELES, CAL., COMP. NOOW
MOTION RELATIVE TO GROUND
ROOF
RELATIVE DISPLACEMENT
INCHES
10.0
0
-10.0
4TH FLOOR
RELATIVE DISPLACEMENT
INCHES
10.0
0
-10.0
0
10
20
30
40
50
TIME - SECONDS

Figure 4.6

RESPONSE SPECTRUM

HOLIDAY INN

8244 ORION BLVD., 1ST FLOOR, LOS ANGELES, CAL., COMP. NOOW

DAMPING VALUES ARE 0, 2, 5, 10 AND 20 PERCENT OF CRITICAL

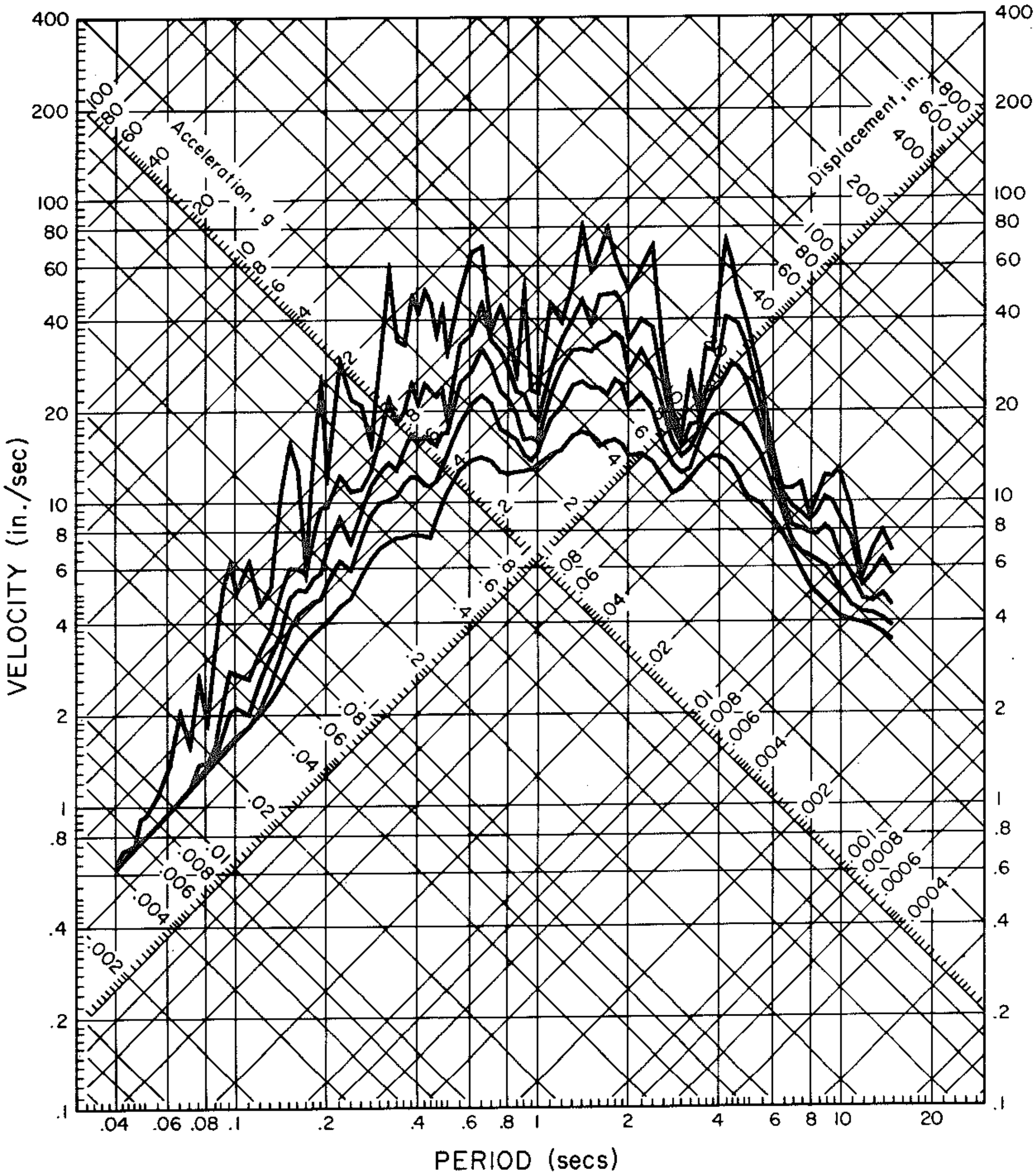


Figure 4.7

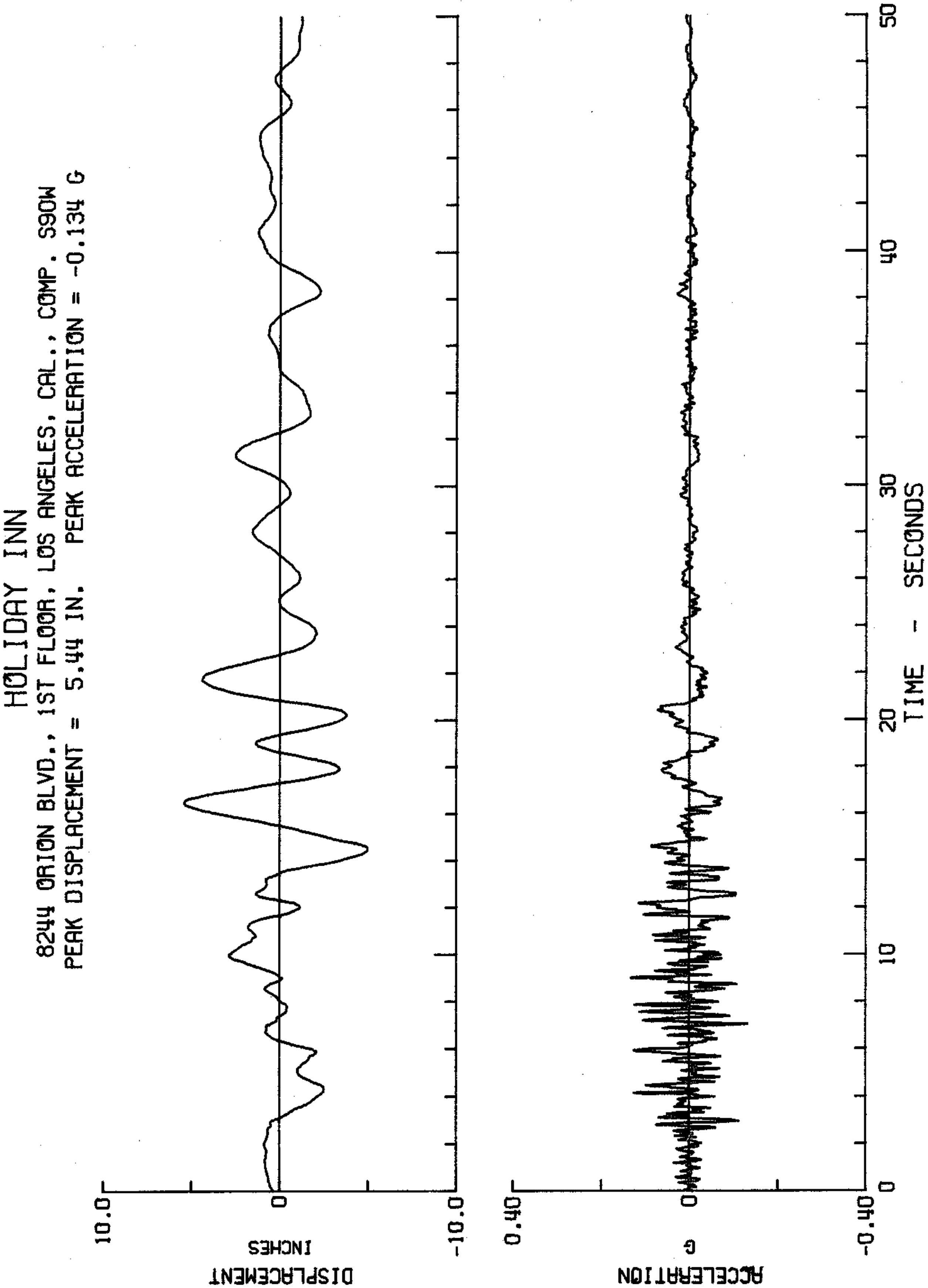

HOLIDAY INN
8244 ORION BLVD., 1ST FLOOR, LOS ANGELES, CAL., COMP. S90W
PEAK DISPLACEMENT = 5.44 IN.   PEAK ACCELERATION = -0.134 G
DISPLACEMENT
INCHES
10.0
0
-10.0
ACCELERATION
G
0.40
0
-0.40
0
10
20
30
40
50
TIME - SECONDS

Figure 4.8

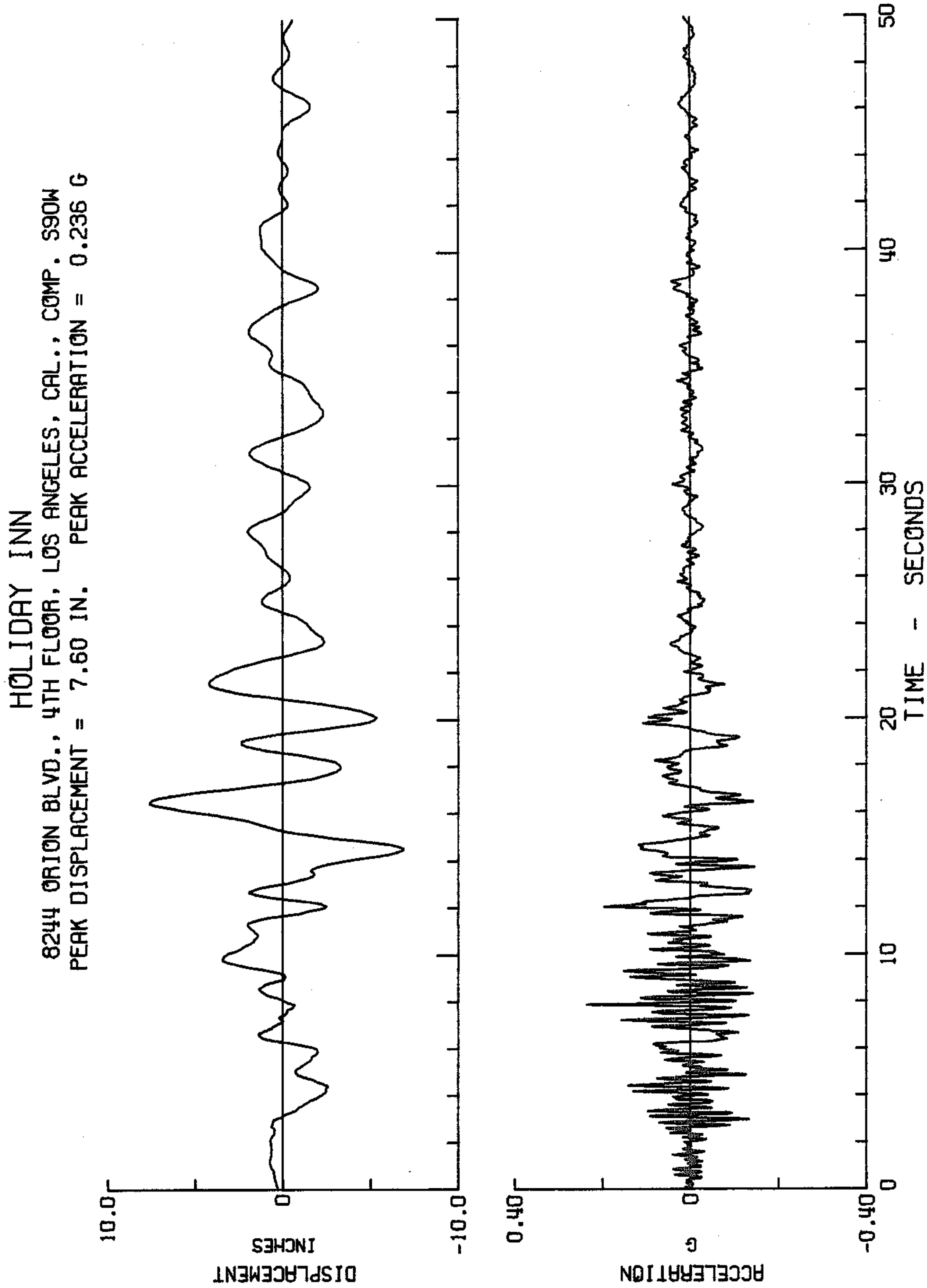

HOLIDAY INN
8244 ORION BLVD., 4TH FLOOR, LOS ANGELES, CAL., COMP. S90W
PEAK DISPLACEMENT = 7.60 IN.
PEAK ACCELERATION = 0.236 G
DISPLACEMENT
INCHES
10.0
0
-10.0
ACCELERATION
G
0.40
0
-0.40
0
10
20
30
40
50
TIME - SECONDS

Figure 4.9

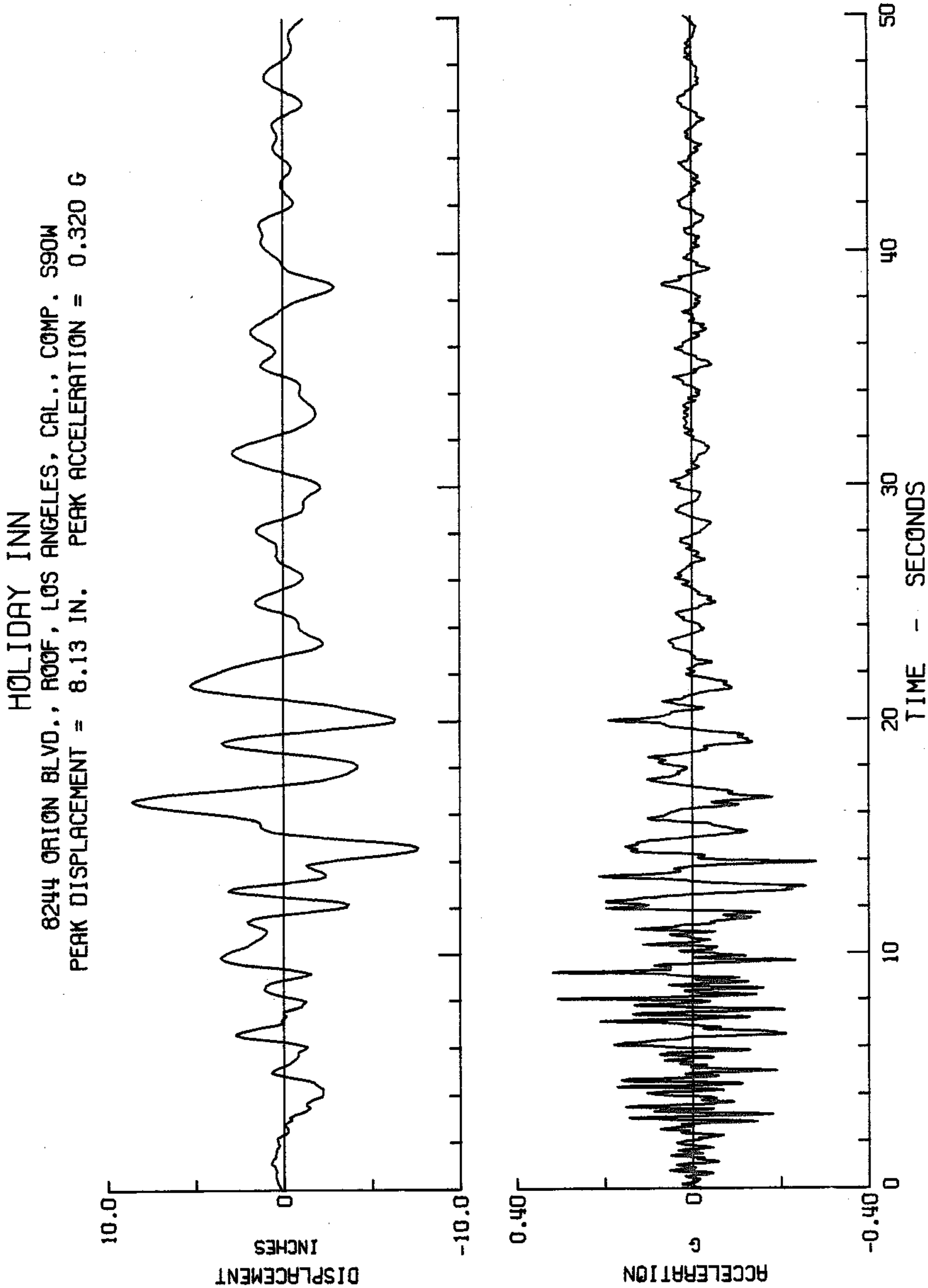
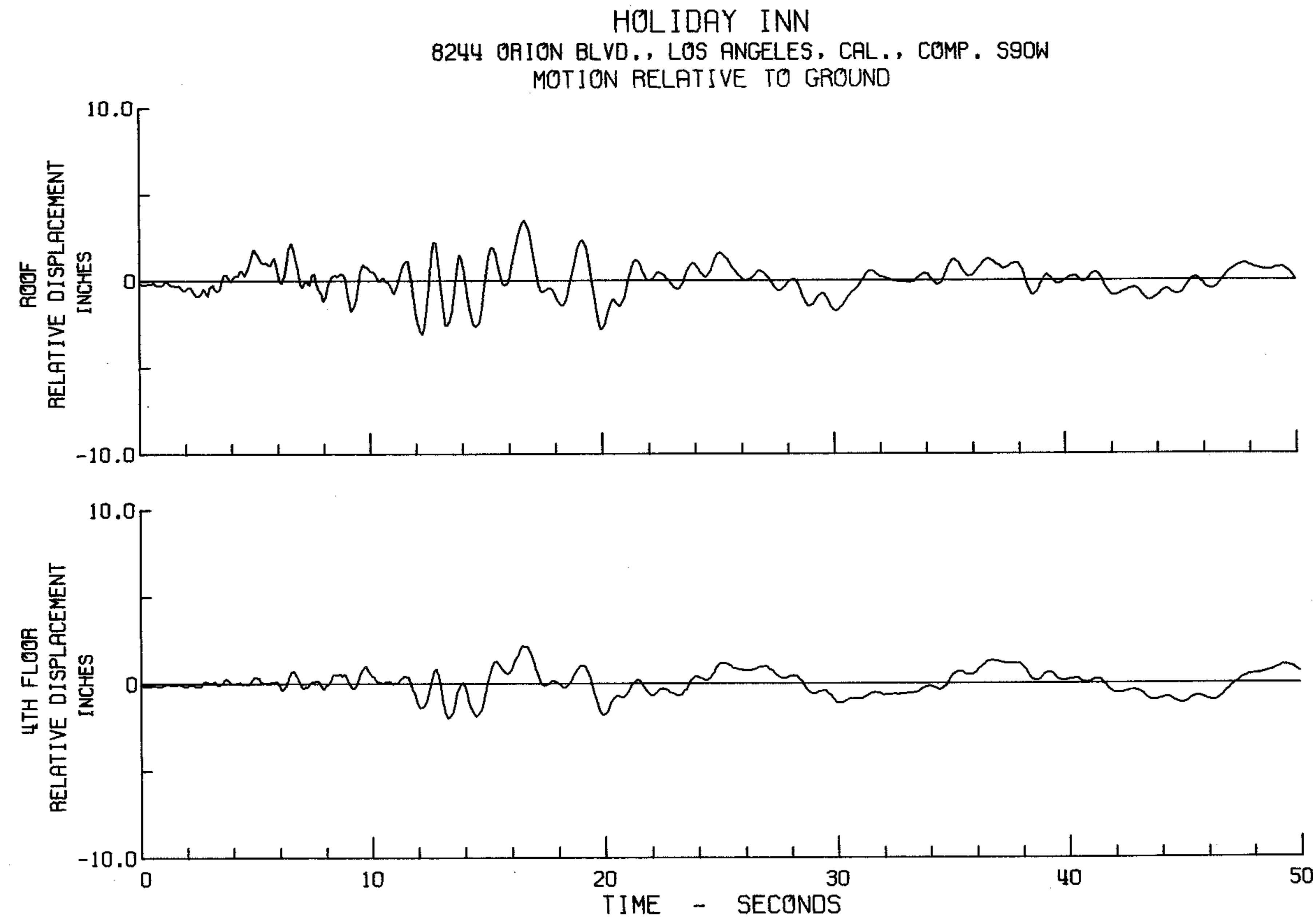

HOLIDAY INN
8244 ORION BLVD., ROOF, LOS ANGELES, CAL., COMP. S90W
PEAK DISPLACEMENT = 8.13 IN.   PEAK ACCELERATION = 0.320 G
DISPLACEMENT
INCHES
10.0
0
-10.0
ACCELERATION
G
0.40
0
-0.40
0
10
20
30
40
50
TIME - SECONDS

Figure 4.10



HOLIDAY INN
8244 ORION BLVD., LOS ANGELES, CAL., COMP. S90W
MOTION RELATIVE TO GROUND
ROOF
RELATIVE DISPLACEMENT
INCHES
10.0
0
-10.0
4TH FLOOR
RELATIVE DISPLACEMENT
INCHES
10.0
0
-10.0
0
10
20
30
40
50
TIME - SECONDS

Figure 4.11

RESPONSE SPECTRUM

HOLIDAY INN

8244 ORION BLVD., 1ST FLOOR, LOS ANGELES, CAL., COMP. S90W

DAMPING VALUES ARE 0, 2, 5, 10 AND 20 PERCENT OF CRITICAL

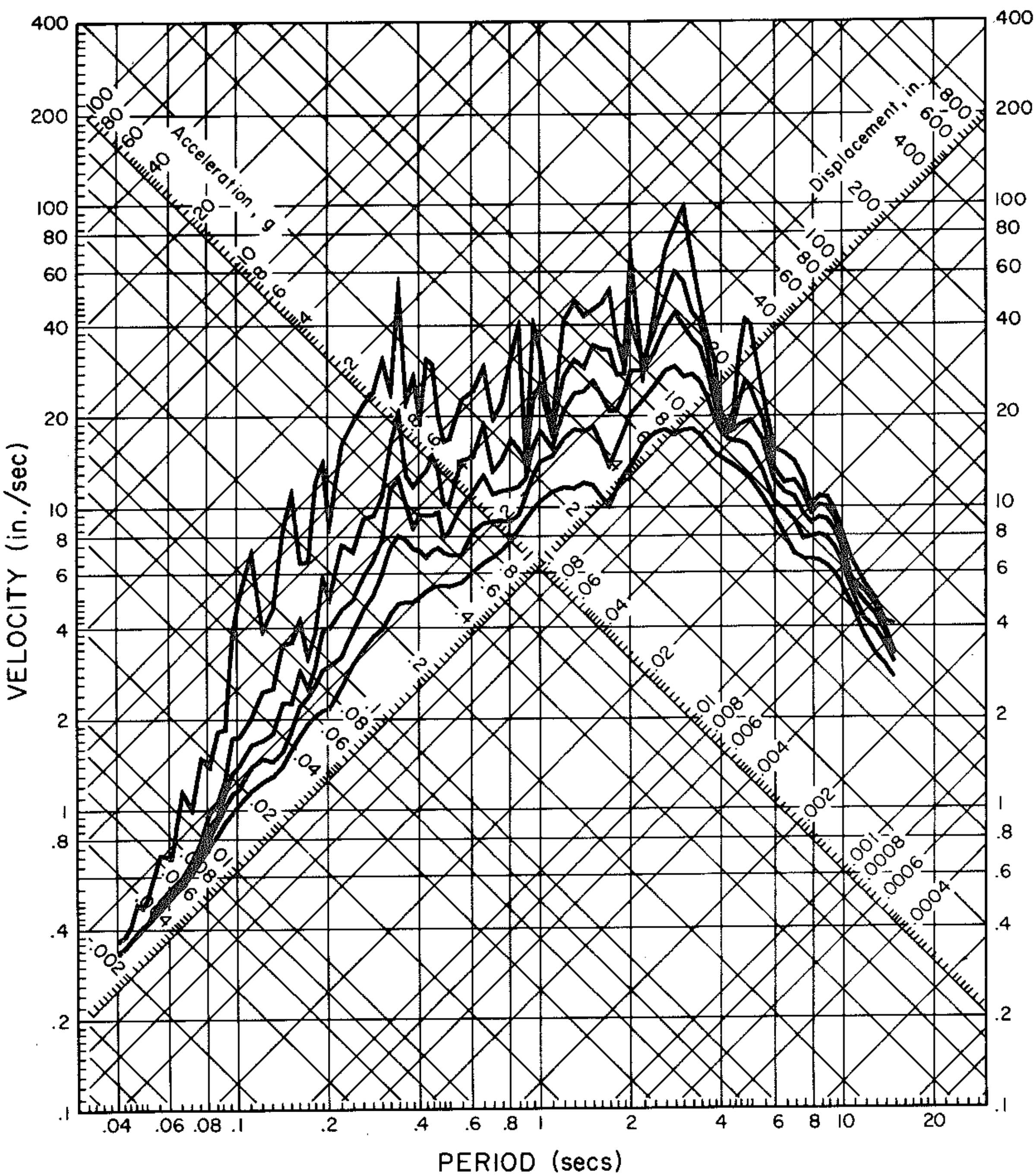


Figure 4.12

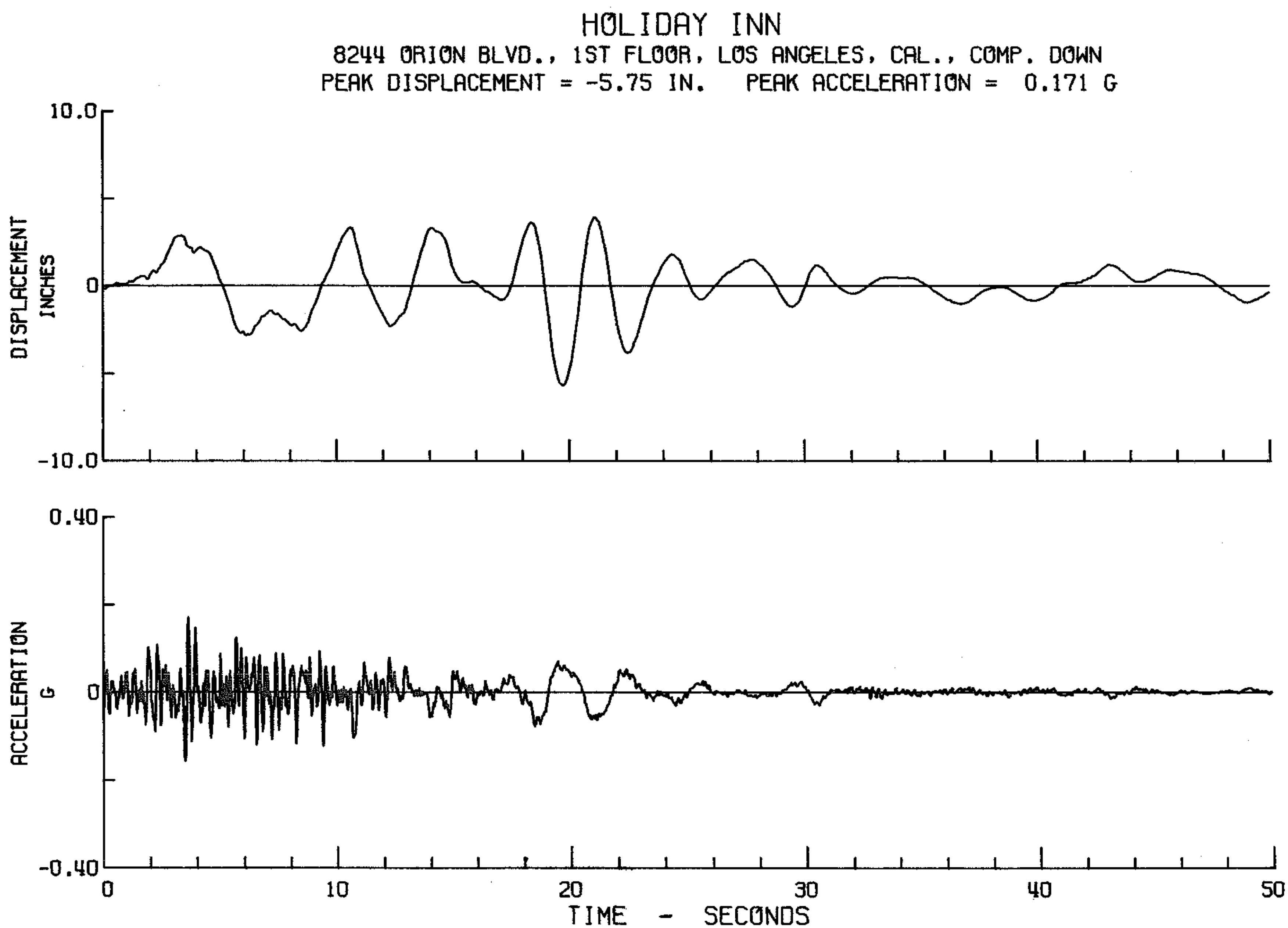

HOLIDAY INN
8244 ORION BLVD., 1ST FLOOR, LOS ANGELES, CAL., COMP. DOWN
PEAK DISPLACEMENT = -5.75 IN.   PEAK ACCELERATION = 0.171 G
DISPLACEMENT
INCHES
10.0
0
-10.0
ACCELERATION
G
0.40
0
-0.40
0
10
20
30
40
50
TIME - SECONDS

Figure 4.13

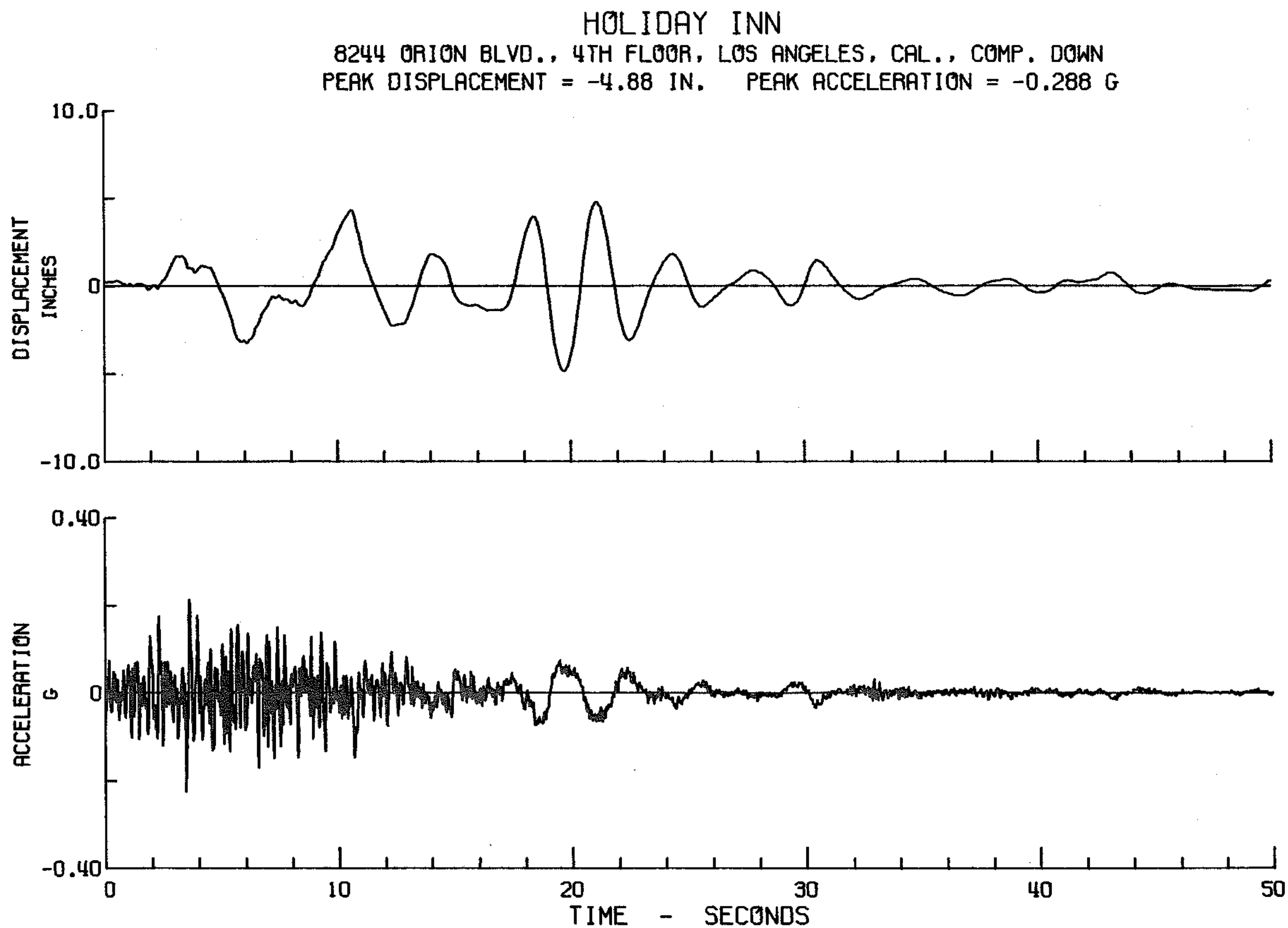

HOLIDAY INN
8244 ORION BLVD., 4TH FLOOR, LOS ANGELES, CAL., COMP. DOWN
PEAK DISPLACEMENT = -4.88 IN.   PEAK ACCELERATION = -0.288 G
10.0
0
-10.0
DISPLACEMENT
INCHES
0.40
0
-0.40
ACCELERATION
G
0
10
20
30
40
50
TIME - SECONDS

Figure 4.14

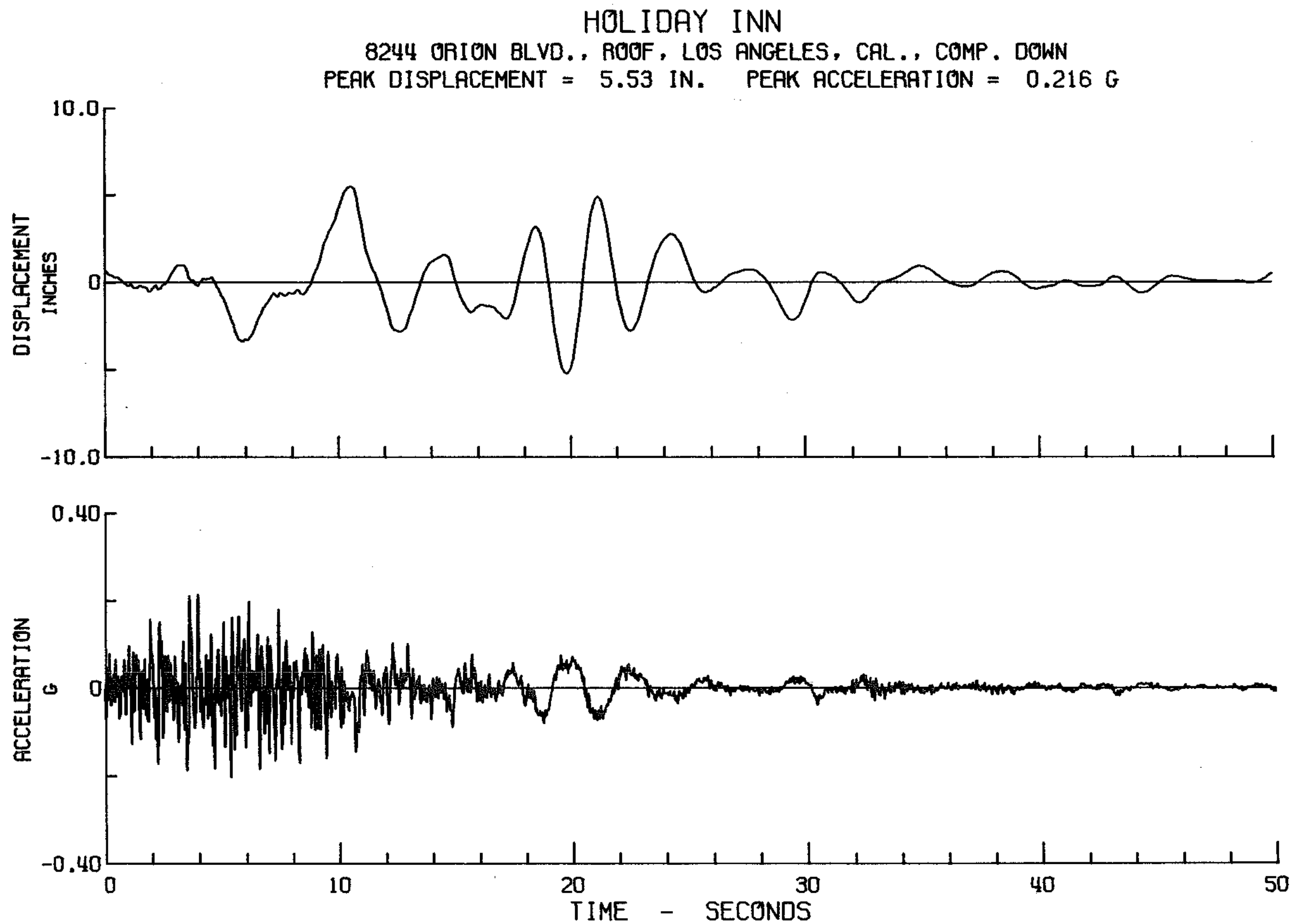

HOLIDAY INN
8244 ORION BLVD., ROOF, LOS ANGELES, CAL., COMP. DOWN
PEAK DISPLACEMENT = 5.53 IN. PEAK ACCELERATION = 0.216 G
DISPLACEMENT INCHES
10.0
0
-10.0
ACCELERATION G
0.40
0
-0.40
0
10
20
30
40
50
TIME - SECONDS

Figure 4.15

RESPONSE SPECTRUM

HOLIDAY INN

8244 ORION BLVD., 1ST FLOOR, LOS ANGELES, CAL., COMP. DOWN

DAMPING VALUES ARE 0, 2, 5, 10 AND 20 PERCENT OF CRITICAL

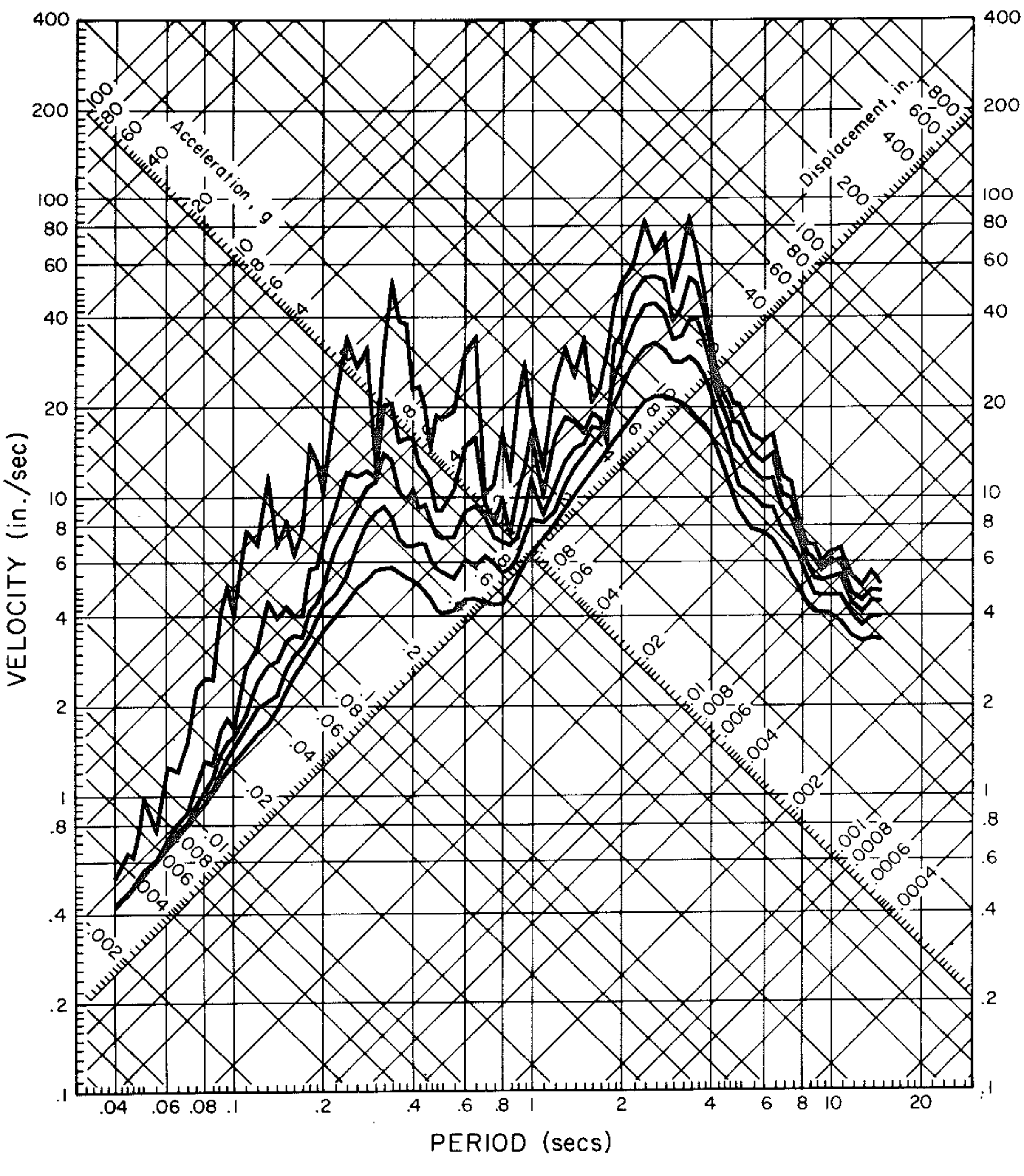


Figure 4.16

Chapter 5

Millikan Library Building
California Institute of Technology
Pasadena, California

The Robert Millikan Memorial Library building, located approximately 19 miles from the center of the San Fernando earthquake, is a nine-story reinforced concrete building constructed in 1966-1967 on the campus of the California Institute of Technology in Pasadena, California. The library building is 69 by 75 feet in plan and extends 144 feet above grade and 158 feet above the basement level. This includes an enclosed roof which houses air conditioning equipment. The basement through ninth floors provide library and office space. Except for the collapse of bookshelves and minor plaster cracking, the building suffered no damage. Figure 5.1 is a picture of the east elevation of Millikan Library.

Lateral loads in the north-south direction are resisted mainly by the 12-inch reinforced concrete shear walls on the east and west ends of the building. In the east-west direction the 12-inch reinforced concrete walls of the central core, which houses the elevator and emergency stairway, provide most of the lateral resistance. Precast concrete grills are bolted in place on the north and south walls. These were intended to be architectural but provide stiffness in the east-west direction for low levels of vibration. The foundation system is composed of a central pad 32 feet wide by 4 feet deep which extends from the east curved wall to the west curved wall. Also provided are beams 10 feet by 2 feet which run east-west beneath the rows of columns at the north and south edges of the building. Stepped beams connect these to the central pad. Figure 5.2a and Figure 5.2b are a transverse section and typical floor plan of Millikan Library.

Recorded acceleration traces and integrated displacements for the vertical, longitudinal and transverse motions recorded in the basement and on the roof of the Millikan Library are given in the following figures. Also shown are the response spectra of the recorded basement accelerations and two horizontal components of relative displacement of the roof.

The north-south and east-west ground accelerations at the Millikan Library building were similar but the ground displacements differed markedly. This, presumably, reflects the orientation of the fault movement relative to the building. In the direction of the large shear walls (N-S) the building vibrated much as would be expected; however, in the east-west direction the vibrations of the building had unexpected characteristics. It can be seen from the seismogram that in the east-west direction the building vibrated with a fundamental mode period of approximately 1.0 second. Before the earthquake, when the building was excited into vibrations by means of a shaking machine, the natural period of vibration of the fundamental east-west mode was 0.66 second, so during the earthquake the period of the fundamental mode was appreciably longer than before the earthquake. Shortly after the earthquake the period was measured and found to be 0.77 second. It is thought that these changes in period represent a change in structural action of the large precast concrete grills that are bolted to the north and south faces of the building. Before the earthquake, during small amplitude vibrations, it is thought that these concrete grills were providing rigidity but this was lost during the large amplitude vibrations produced by the earthquake. Also, the foundation conditions were difficult in the east-west direction than in the north-south direction and possibly some foundation action could have changed during the earthquake.

References

1. Jennings, P. C. and Kuroiwa, J. H., "Vibration and Soil Structure Interaction Tests of a Nine-Story Reinforced Concrete Building", Bull. Seis. Soc. of Amer., Vol. 58, No. 3, June 1968.

2. Udwadia, F. E., and Trifunac, M. D., "Ambient Vibration Tests of Full Scale Structures", Proc. Fifth World Conference on Earthquake Engineering, Rome, 1974.

3. Iemura, H. and Jennings, P. C., "Hysteretic Response of a Nine-Story Reinforced Concrete Building", International Journal of Earthquake Engineering and Structural Dynamics, 3, No. 2, 183 - 202, 1974.

4. Trifunac, M. D., "Comparisons Between Ambient and Forced Vibration Experiments", International Journal of Earthquake Engineering and Structural Dynamics, 3, No. 2, 183 - 202, 1974.

5. Trifunac, M. D. and Udwadia, F. E., "Time and Amplitude Dependent Response of Structures", International Journal of Earthquake Engineering and Structural Dynamics, 2, 359 - 378, 1974.

6. Foutch, D. A., Luco, J. E., Trifunac, M. D., and Udwadia, F. E., "Full Scale, Three-Dimensional Tests of Structural Deformations During Forced Excitation of a Nine-Story Reinforced Concrete Building", Proc. U. S. National Conference on Earthquake Engineering, Ann Arbor, Mich., June, 1975.

Figure 5.1 Millikan Library

⌖ — Location of Strong Motion Instruments

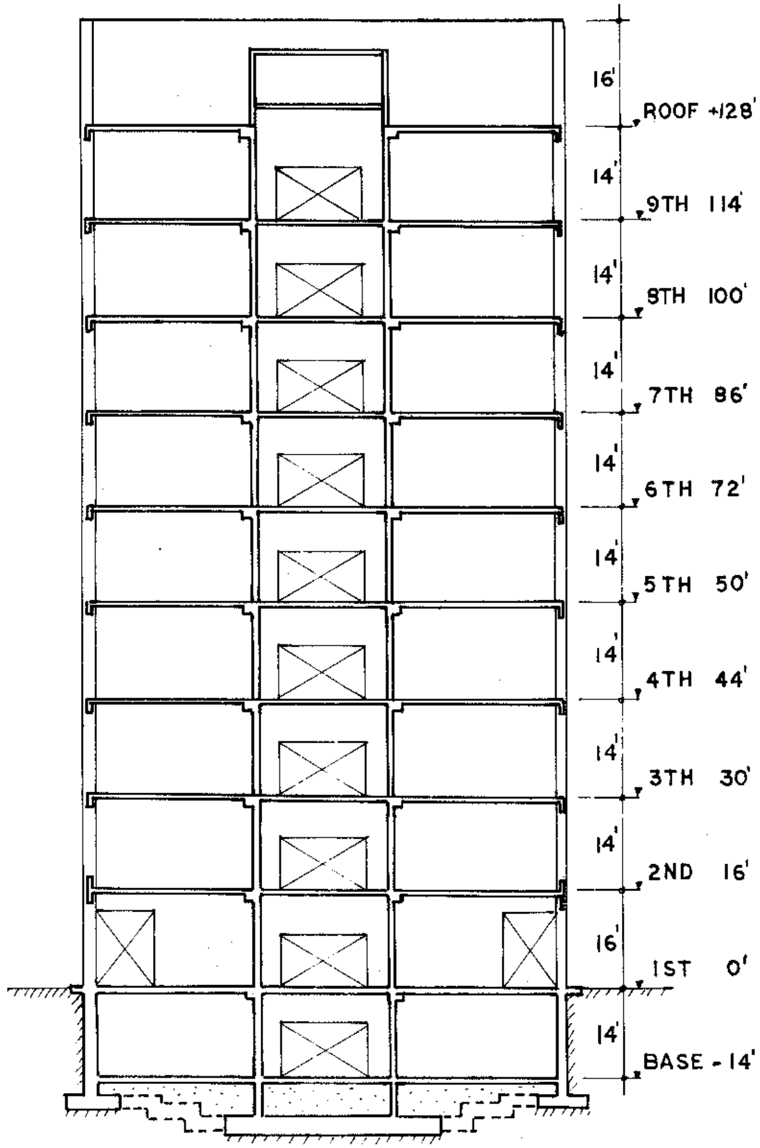


Figure 5.2a Transverse Section

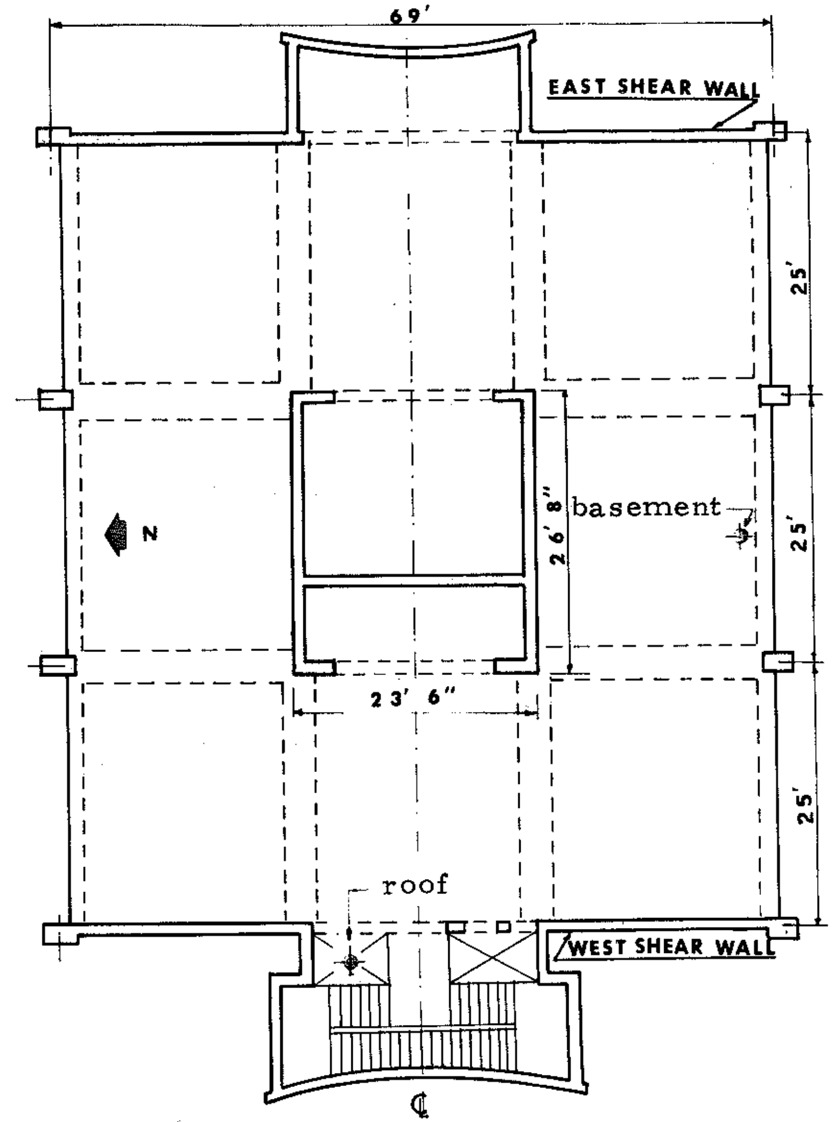


Figure 5.2b Typical Floor Plan

Figure 5.2 Schematic of Millikan Structural System

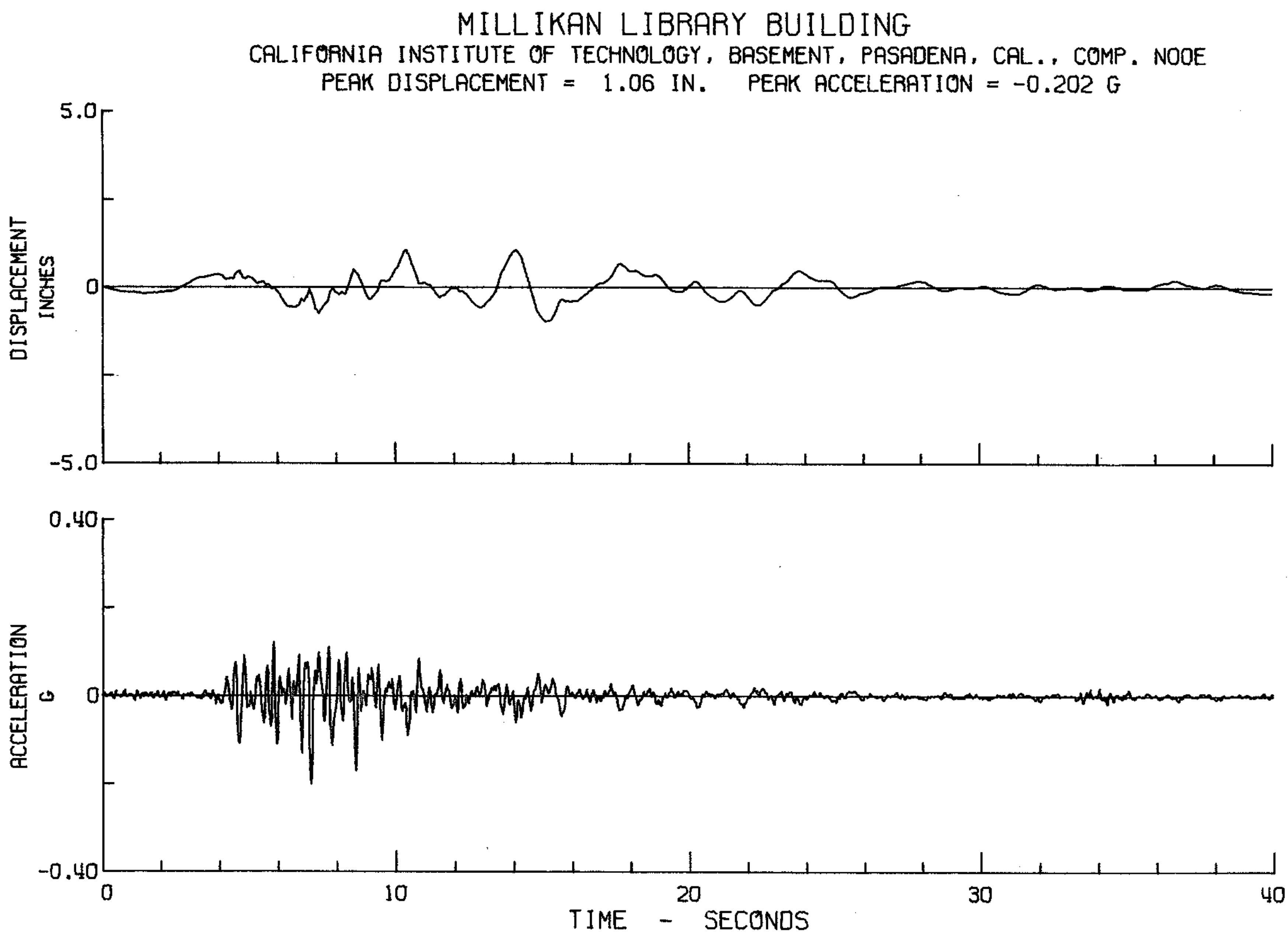

MILLIKAN LIBRARY BUILDING
CALIFORNIA INSTITUTE OF TECHNOLOGY, BASEMENT, PASADENA, CAL., COMP. N00E
PEAK DISPLACEMENT = 1.06 IN. PEAK ACCELERATION = -0.202 G
5.0
0
-5.0
DISPLACEMENT
INCHES
0.40
0
-0.40
ACCELERATION
G
0
10
20
30
40
TIME - SECONDS

Figure 5.3

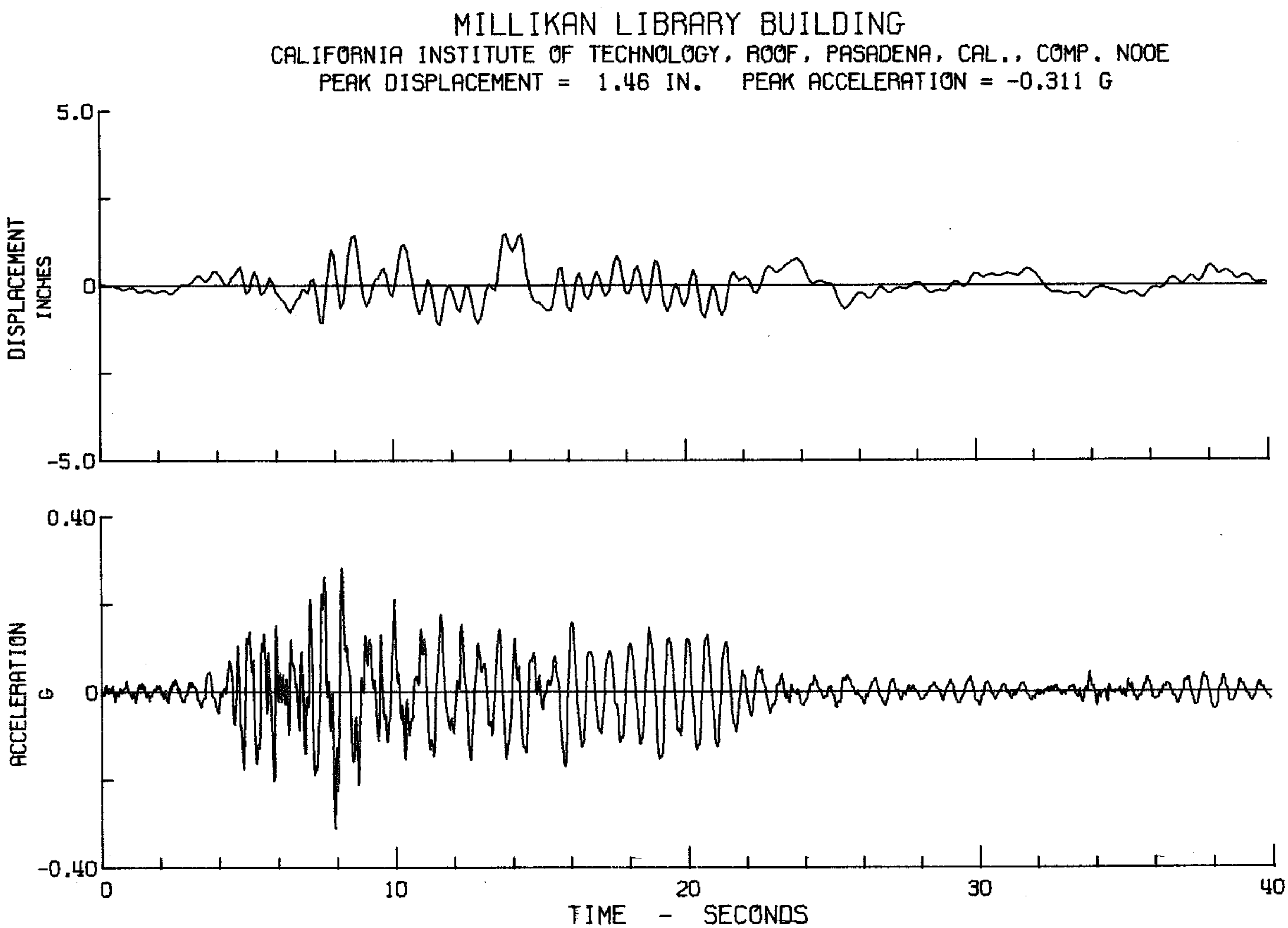

MILLIKAN LIBRARY BUILDING
CALIFORNIA INSTITUTE OF TECHNOLOGY, ROOF, PASADENA, CAL., COMP. NOOE
PEAK DISPLACEMENT = 1.46 IN. PEAK ACCELERATION = -0.311 G
DISPLACEMENT
INCHES
5.0
0
-5.0
ACCELERATION
G
0.40
0
-0.40
0
10
20
30
40
TIME - SECONDS

Figure 5.4

MILLIKAN LIBRARY BUILDING
CALIFORNIA INSTITUTE OF TECHNOLOGY, PASADENA, CAL., COMP. N00E
MOTION RELATIVE TO GROUND

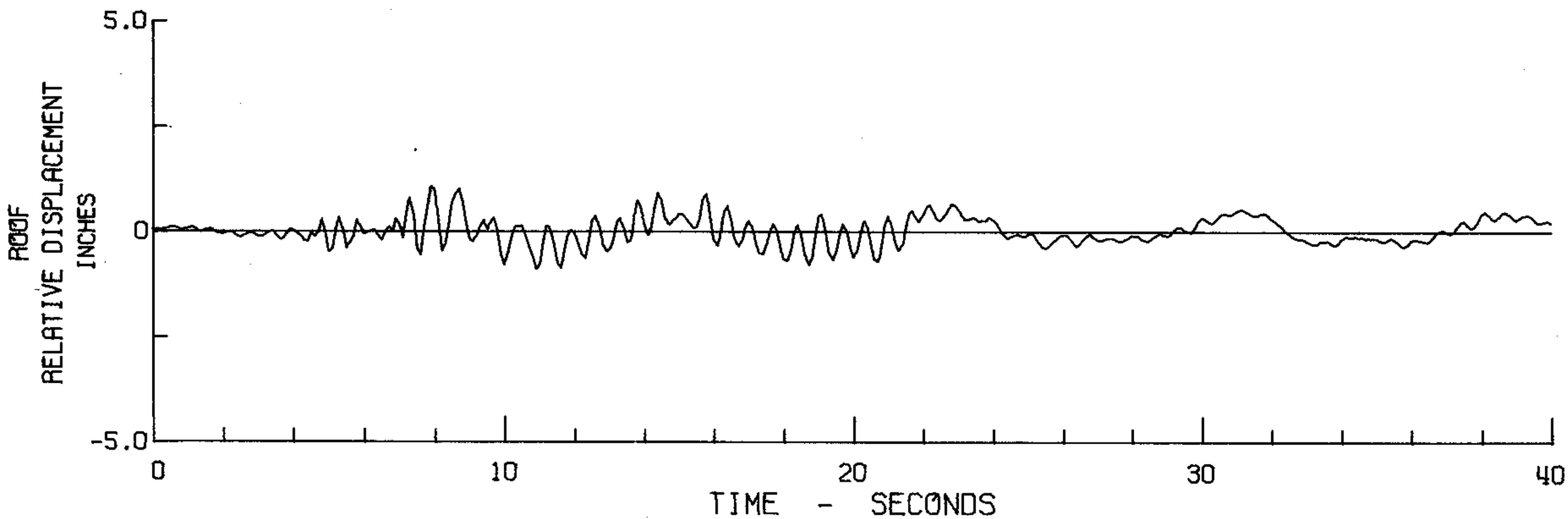


Figure 5.5

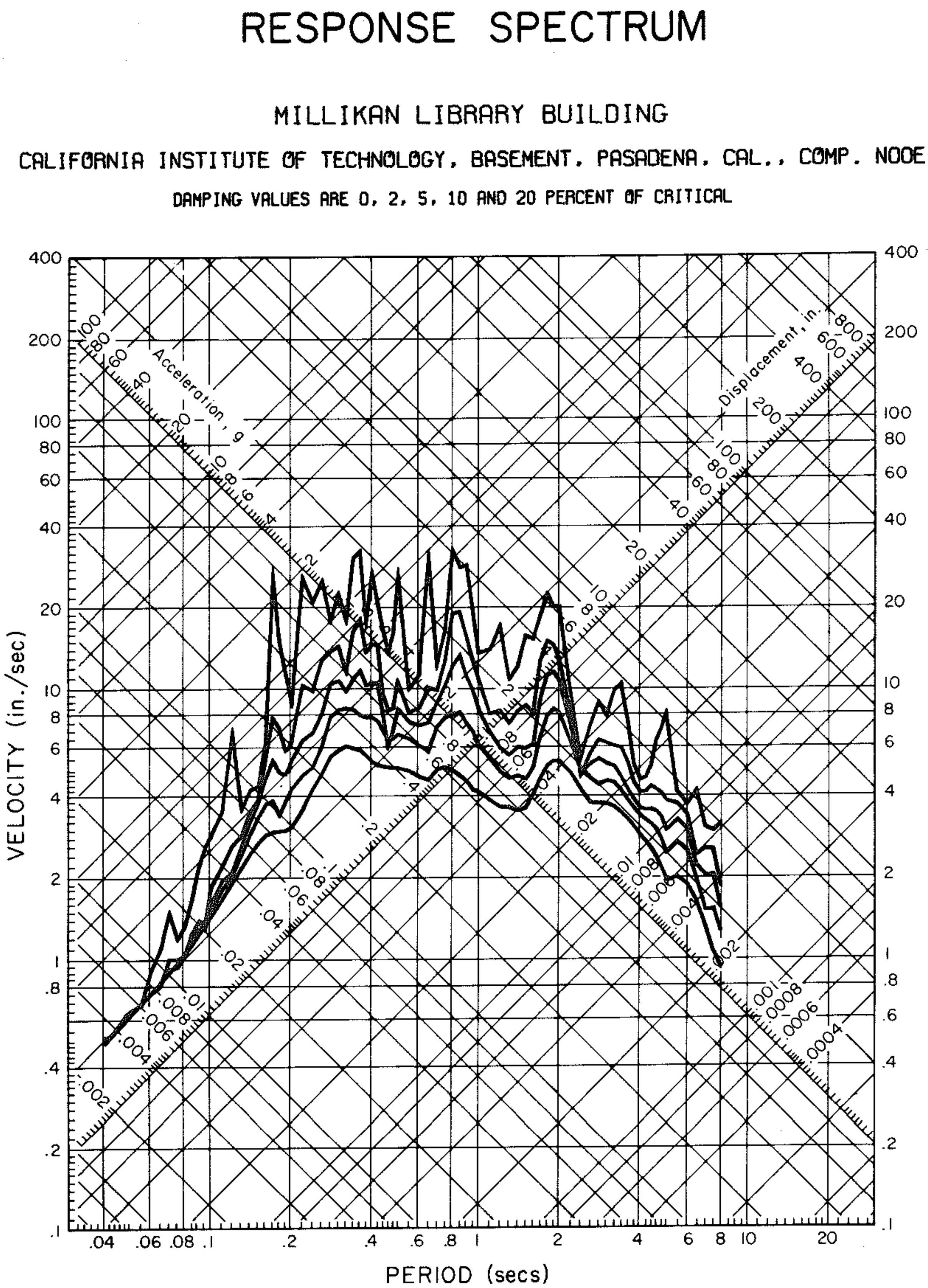

RESPONSE SPECTRUM
MILLIKAN LIBRARY BUILDING
CALIFORNIA INSTITUTE OF TECHNOLOGY, BASEMENT, PASADENA, CAL., COMP. NODE
DAMPING VALUES ARE 0, 2, 5, 10 AND 20 PERCENT OF CRITICAL
VELOCITY (in./sec)
PERIOD (secs)
Acceleration, g
Displacement, in.

Figure 5.6

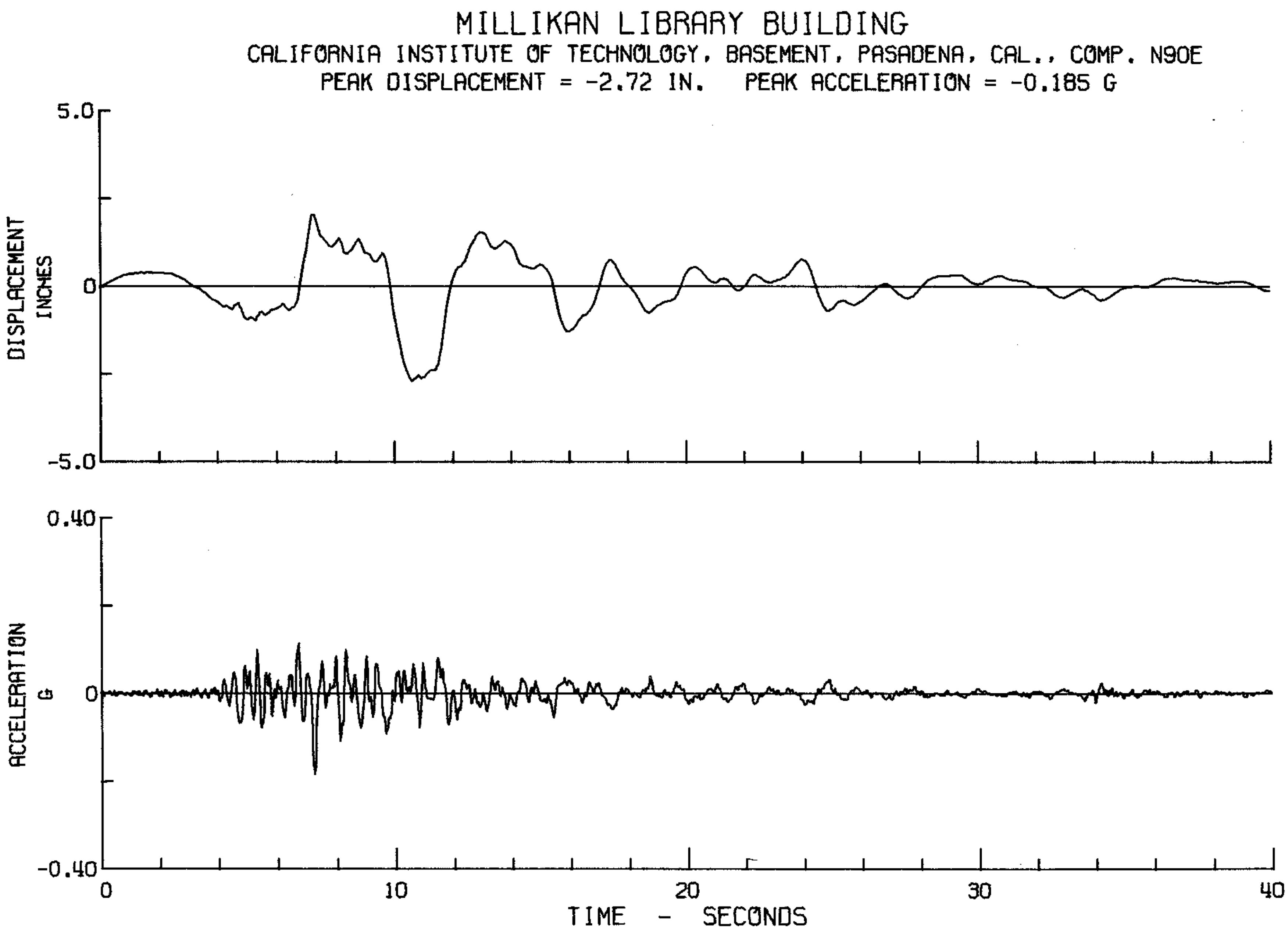

MILLIKAN LIBRARY BUILDING
CALIFORNIA INSTITUTE OF TECHNOLOGY, BASEMENT, PASADENA, CAL., COMP. N90E
PEAK DISPLACEMENT = -2.72 IN.   PEAK ACCELERATION = -0.185 G
DISPLACEMENT
INCHES
5.0
0
-5.0
ACCELERATION
G
0.40
0
-0.40
0
10
20
30
40
TIME - SECONDS

Figure 5.7

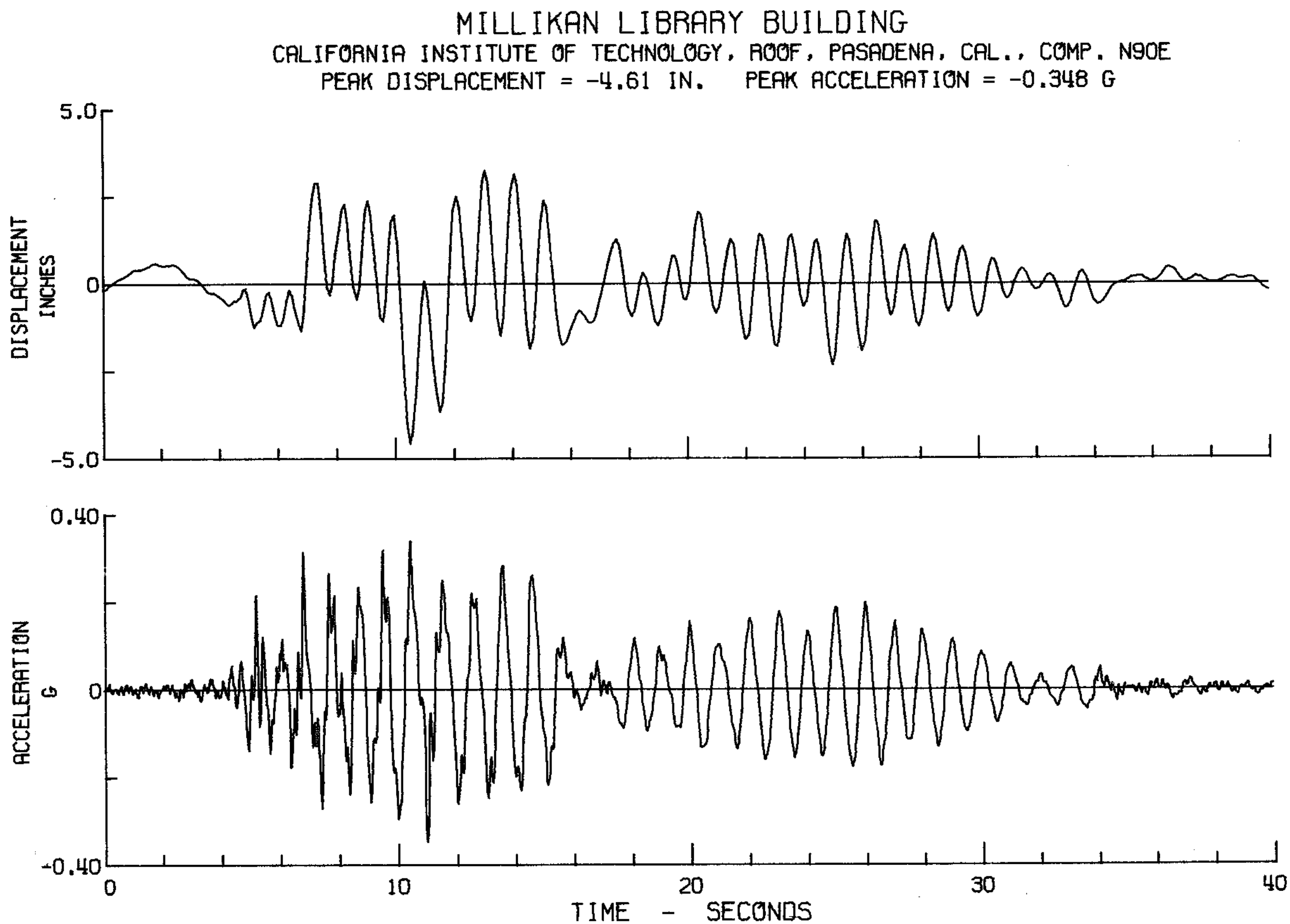

MILLIKAN LIBRARY BUILDING
CALIFORNIA INSTITUTE OF TECHNOLOGY, ROOF, PASADENA, CAL., COMP. N90E
PEAK DISPLACEMENT = -4.61 IN.   PEAK ACCELERATION = -0.348 G
DISPLACEMENT
INCHES
5.0
0
-5.0
ACCELERATION
G
0.40
0
-0.40
0
10
20
30
40
TIME - SECONDS

Figure 5.8

MILLIKAN LIBRARY BUILDING
CALIFORNIA INSTITUTE OF TECHNOLOGY, PASADENA, CAL., COMP. N90E
MOTION RELATIVE TO GROUND

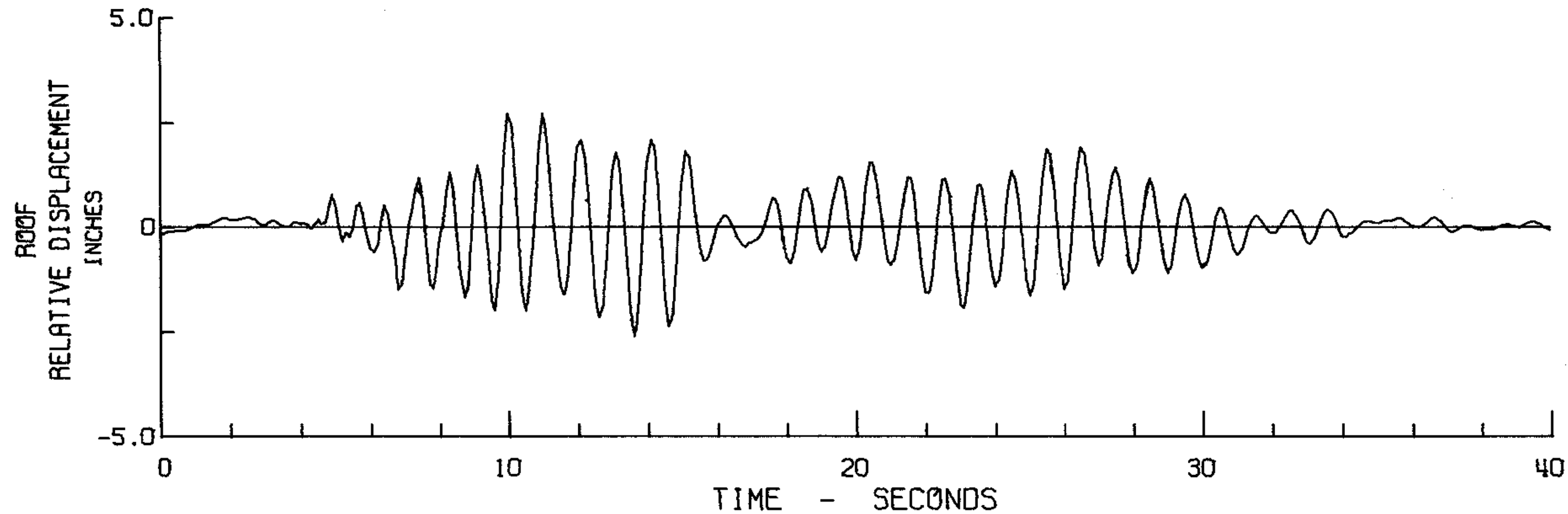


Figure 5.9

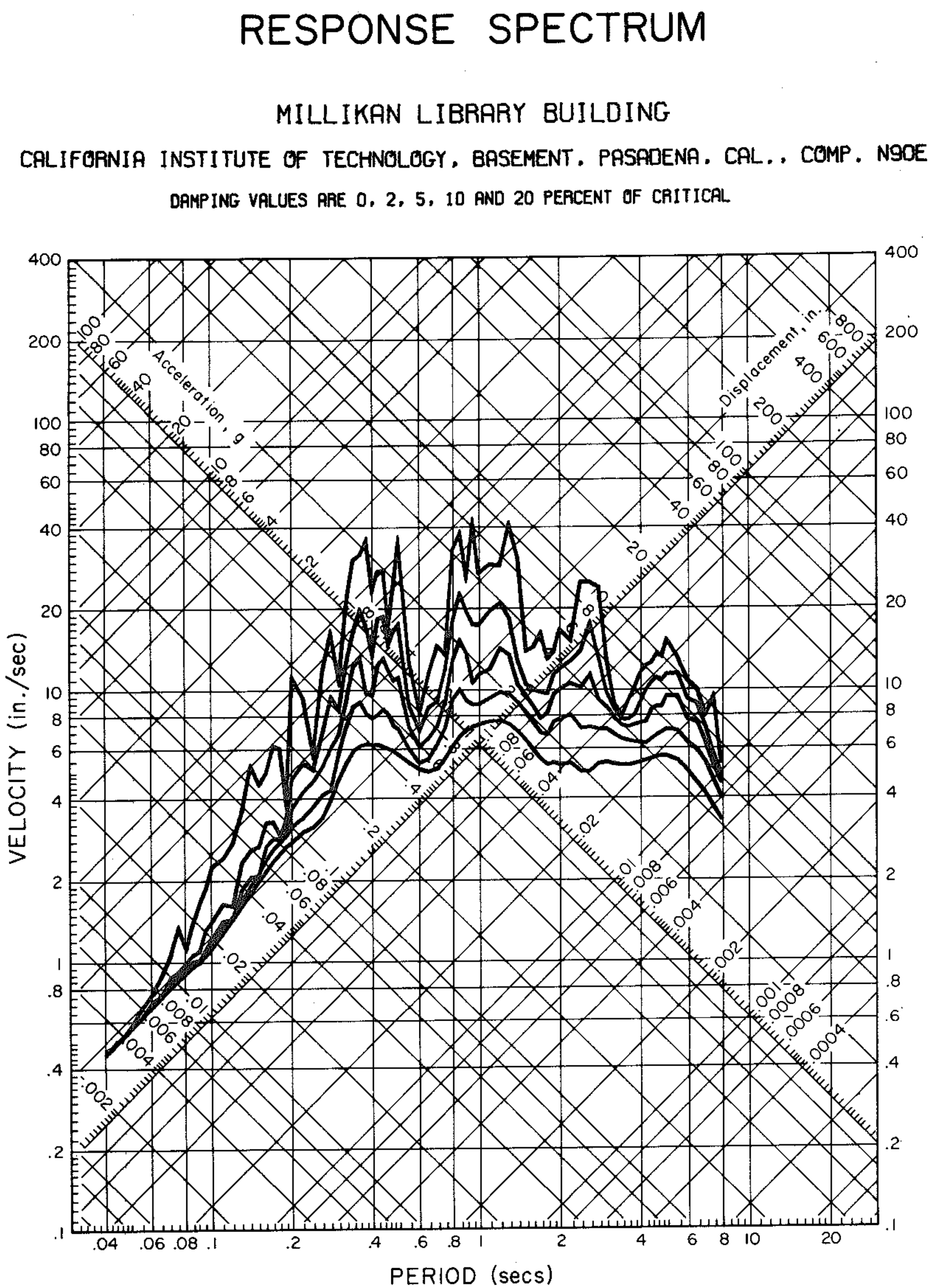

RESPONSE SPECTRUM
MILLIKAN LIBRARY BUILDING
CALIFORNIA INSTITUTE OF TECHNOLOGY, BASEMENT, PASADENA, CAL., COMP. N90E
DAMPING VALUES ARE 0, 2, 5, 10 AND 20 PERCENT OF CRITICAL
Acceleration, g
Displacement, in.
VELOCITY (in./sec)
PERIOD (secs)

Figure 5.10

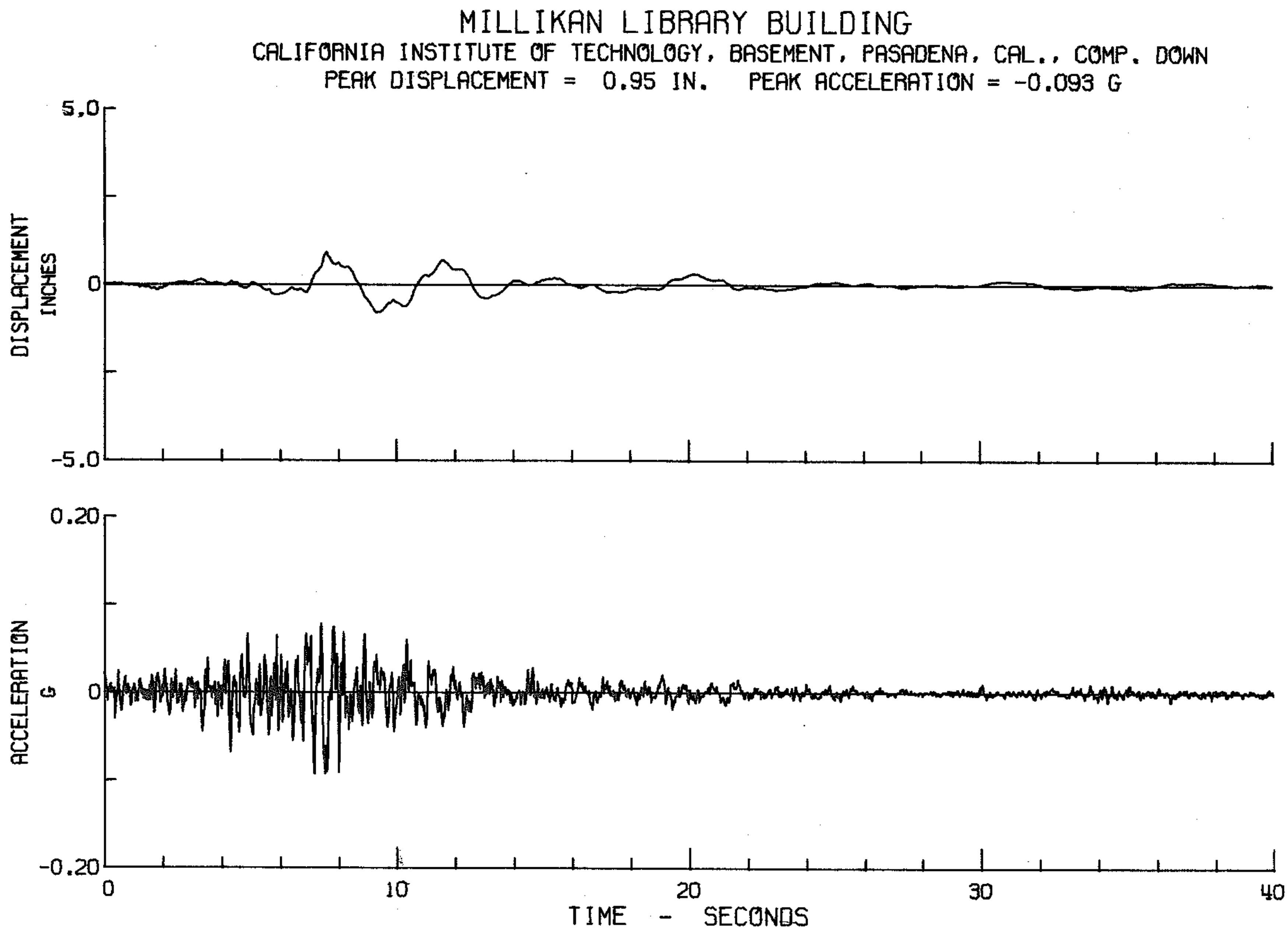

MILLIKAN LIBRARY BUILDING
CALIFORNIA INSTITUTE OF TECHNOLOGY, BASEMENT, PASADENA, CAL., COMP. DOWN
PEAK DISPLACEMENT = 0.95 IN. PEAK ACCELERATION = -0.093 G
5.0
0
-5.0
DISPLACEMENT
INCHES
0.20
0
-0.20
ACCELERATION
G
0
10
20
30
40
TIME - SECONDS

Figure 5.11

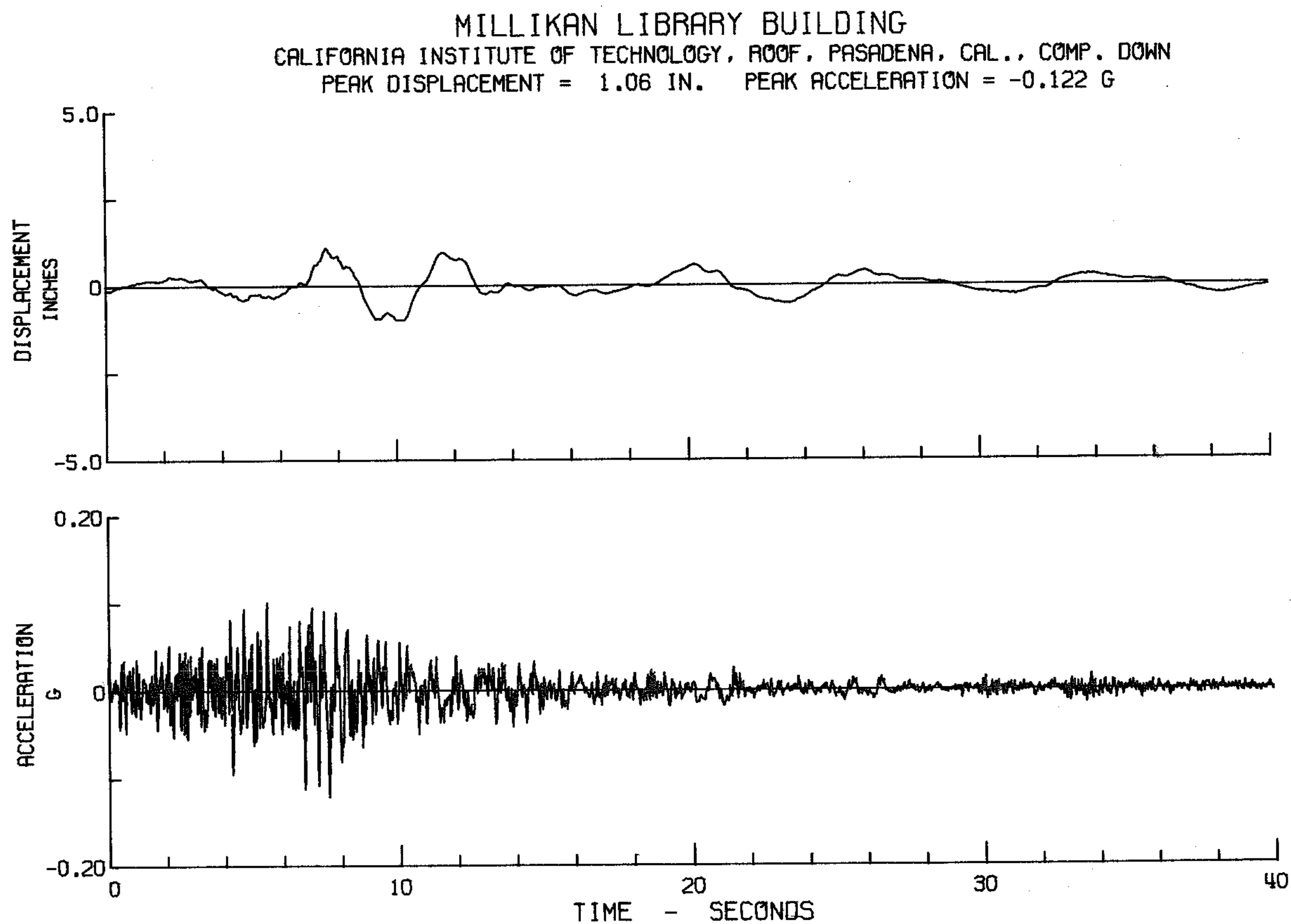

MILLIKAN LIBRARY BUILDING
CALIFORNIA INSTITUTE OF TECHNOLOGY, ROOF, PASADENA, CAL., COMP. DOWN
PEAK DISPLACEMENT = 1.06 IN. PEAK ACCELERATION = -0.122 G
DISPLACEMENT
INCHES
5.0
0
-5.0
ACCELERATION
G
0.20
0
-0.20
0
10
20
30
40
TIME - SECONDS

Figure 5.12

RESPONSE SPECTRUM

MILLIKAN LIBRARY BUILDING

CALIFORNIA INSTITUTE OF TECHNOLOGY, BASEMENT, PASADENA, CAL., COMP. DOWN

DAMPING VALUES ARE 0, 2, 5, 10 AND 20 PERCENT OF CRITICAL

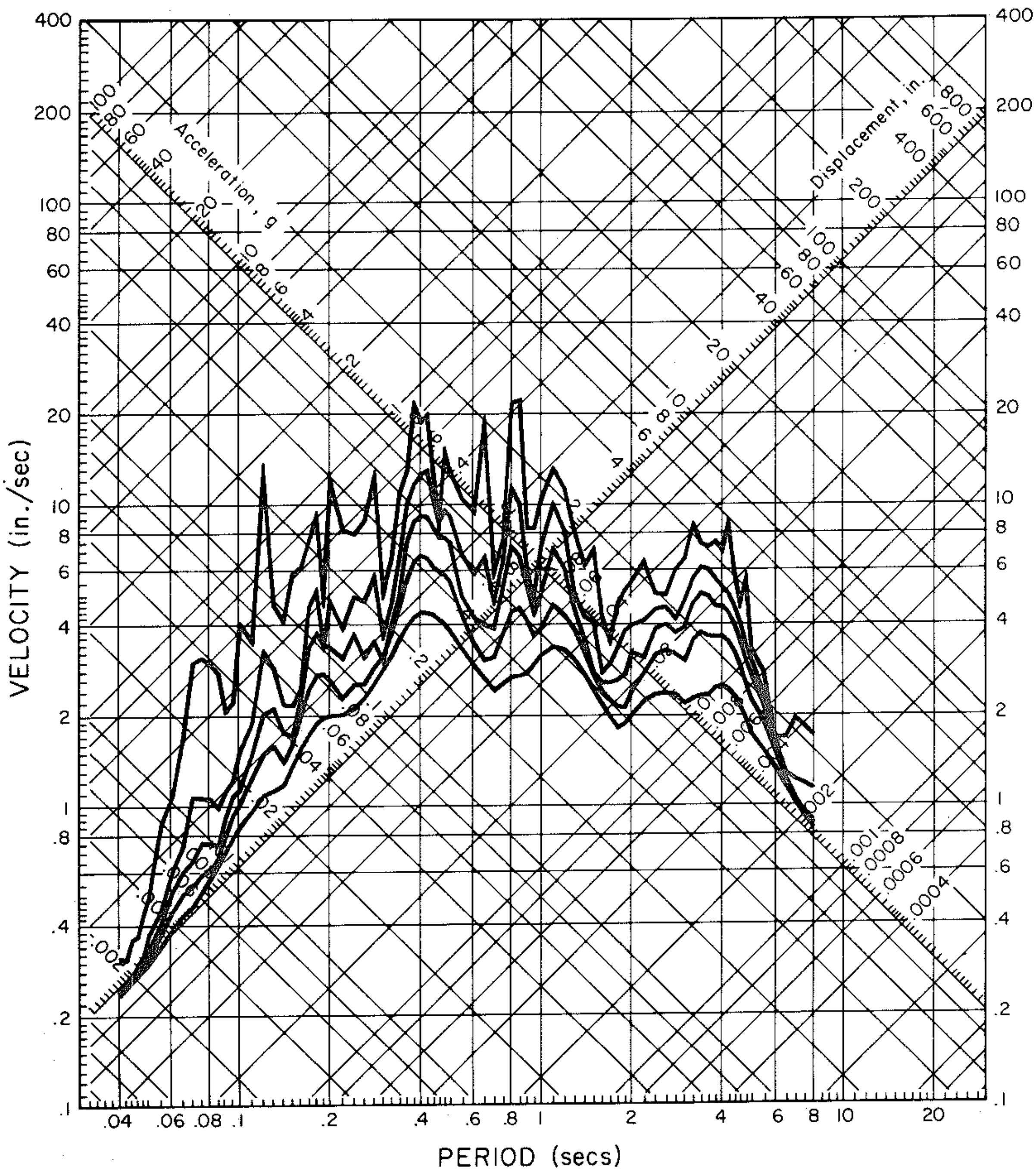


Figure 5.13

Chapter 6

Building 180
Jet Propulsion Laboratory
Pasadena, California

Building 180, located approximately 15 miles from the center of the San Fernando earthquake, serves as an administration building for the Jet Propulsion Laboratory. The nine-story steel frame structure was designed in 1961. The plan dimensions of the building are 40 by 220 feet. The height of the building is 146 feet from foundation to roof with 114 feet above grade on the north side and 130 feet above grade on the south side. The building suffered no structural damage and only minor nonstructural cracking. Figure 6.1 is a picture of the south face of Building 180.

The lateral load resisting system of Building 180 is atypical for southern California. Lateral loads are resisted in the transverse direction by long span welded steel spandrel trusses acting as girders and steel columns partially encased in concrete. In the longitudinal direction the lateral load resisting frame is composed of deep steel girders and steel columns. Continuous strip footings in the longitudinal direction serve as the foundation system. Figure 6.2 shows typical transverse and longitudinal sections as well as a typical plan of the structural system of Building 180.

Each of the strong motion instruments located at the basement and roof of Building 180 recorded two horizontal and one vertical component of acceleration during the San Fernando earthquake. The acceleration traces and their integrated displacement records are shown in the following figures. Also shown are the response spectra for the three components of the basement motion as well as the two horizontal components of the relative displacement of the roof motion.

The seismograms recorded in the basement of the Jet Propulsion Laboratory building show a marked difference in the long period displacement just as was the case in the Millikan Library building. The peak roof acceleration, in the S82°E direction was approximately 40% g, which resulted from a strong excitation of the 2nd mode of vibration. Dynamic analyses reported in reference 1 showed that the peak stresses in the structural frame did not quite reach the yield point during the earthquake.

References

1. Wood, J.H., Analysis of the Earthquake Response of a Nine-Story Steel Frame Building During the San Fernando Earthquake, Earthquake Engineering Research Laboratory Report No. 72-04, California Institute of Technology, Pasadena, California, 1971.

2. Nielsen, N.N., "Vibration Tests of a Nine-Story Steel Frame Building", Proc., ASCE, Journal of Applied Mech., Vol. 92, No. EM1., February, 1966, pp. 81 - 110.

3. Brandow and Johnston Associates, "Design Analysis of the Jet Propulsion Laboratory, Building 180", prepared for the Jet Propulsion Laboratory, California Institute of Technology, Pasadena, California, 1971.

Figure 6.1 Building 180, Jet Propulsion Laboratory

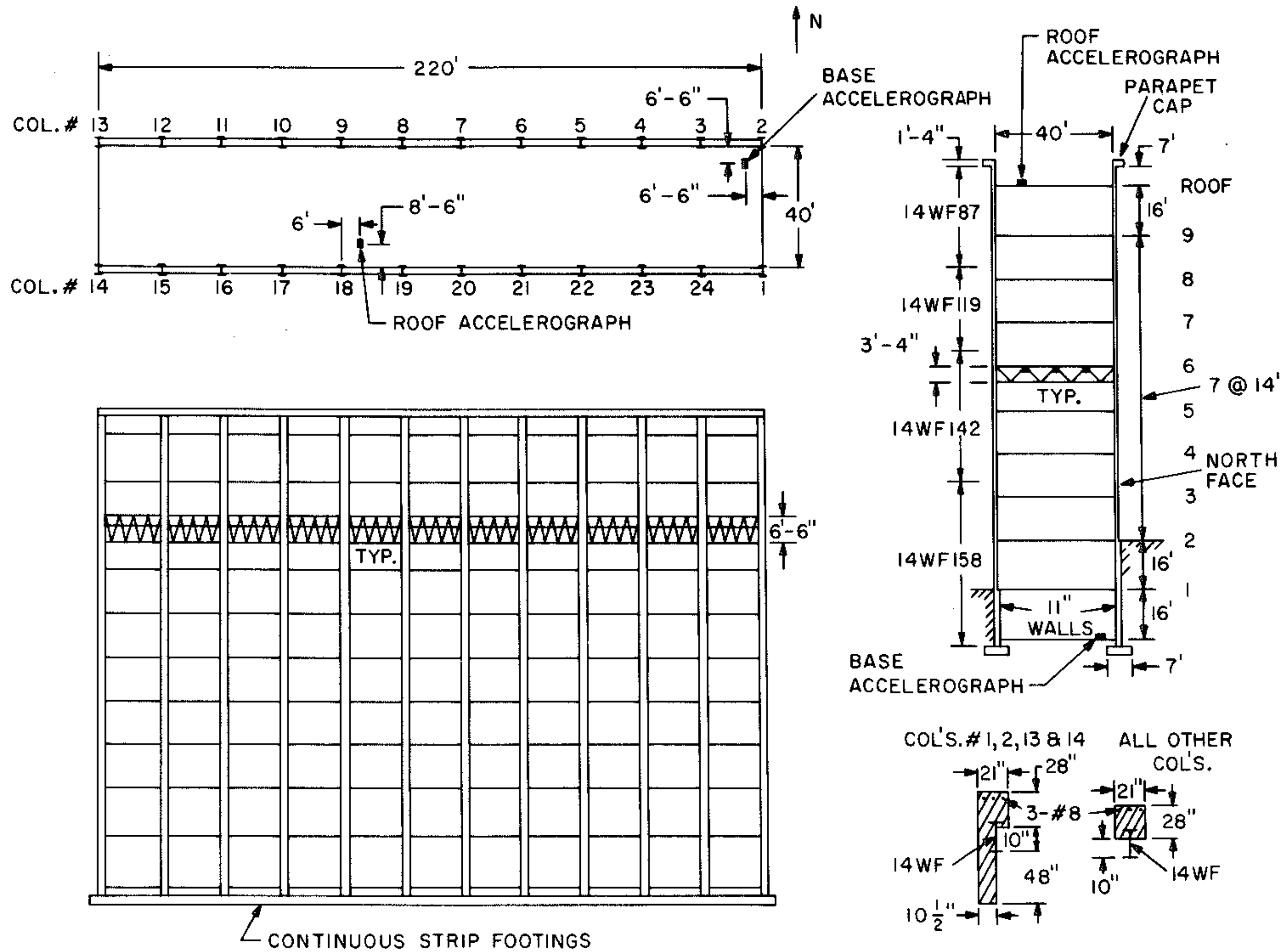


Figure 6.2 Longitudinal and Transverse Sections and Floor Plan of JPL Building 180.

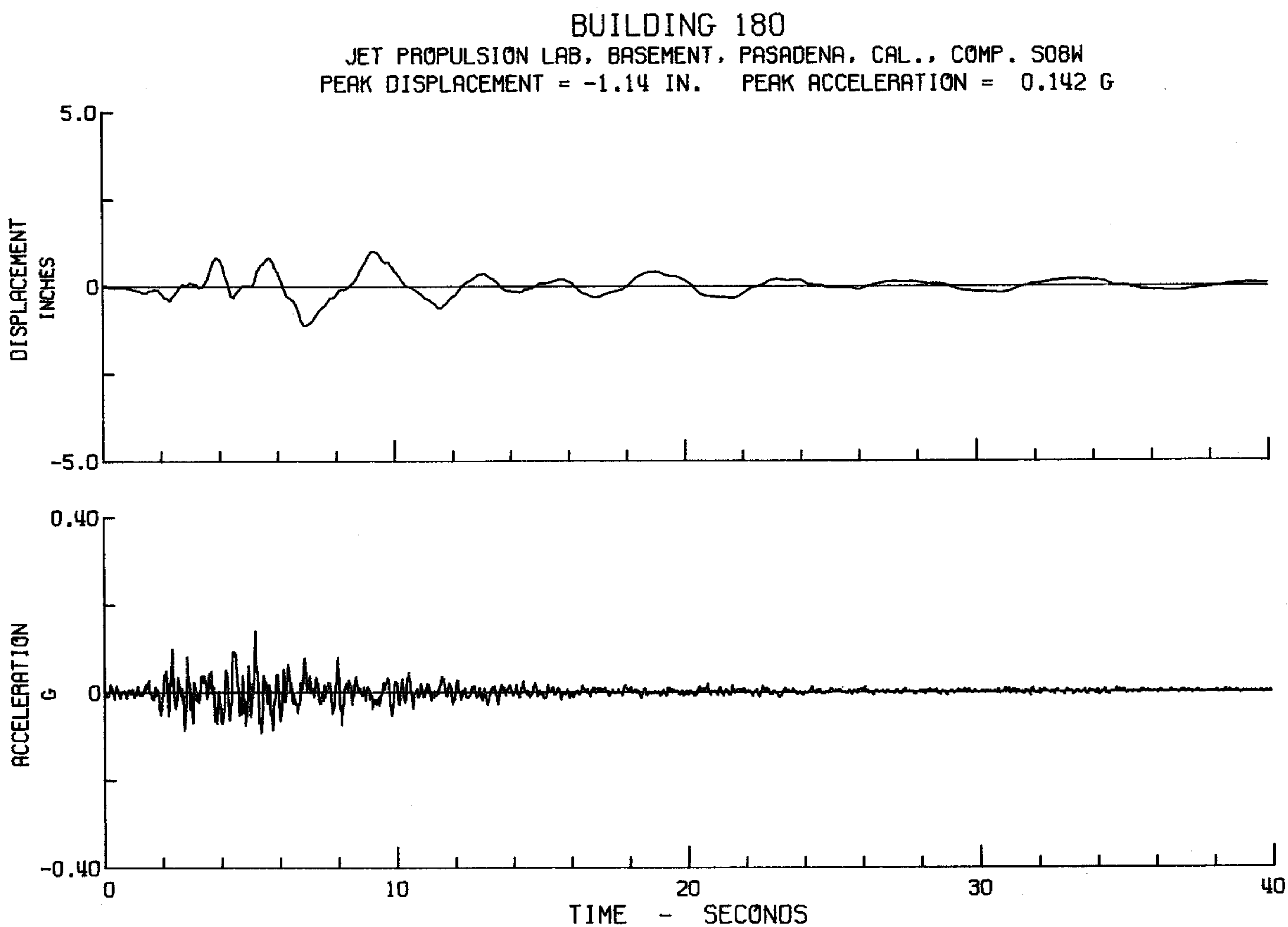

BUILDING 180
JET PROPULSION LAB, BASEMENT, PASADENA, CAL., COMP. S08W
PEAK DISPLACEMENT = -1.14 IN.   PEAK ACCELERATION = 0.142 G
DISPLACEMENT INCHES
5.0
0
-5.0
ACCELERATION G
0.40
0
-0.40
0
10
20
30
40
TIME - SECONDS

Figure 6.3

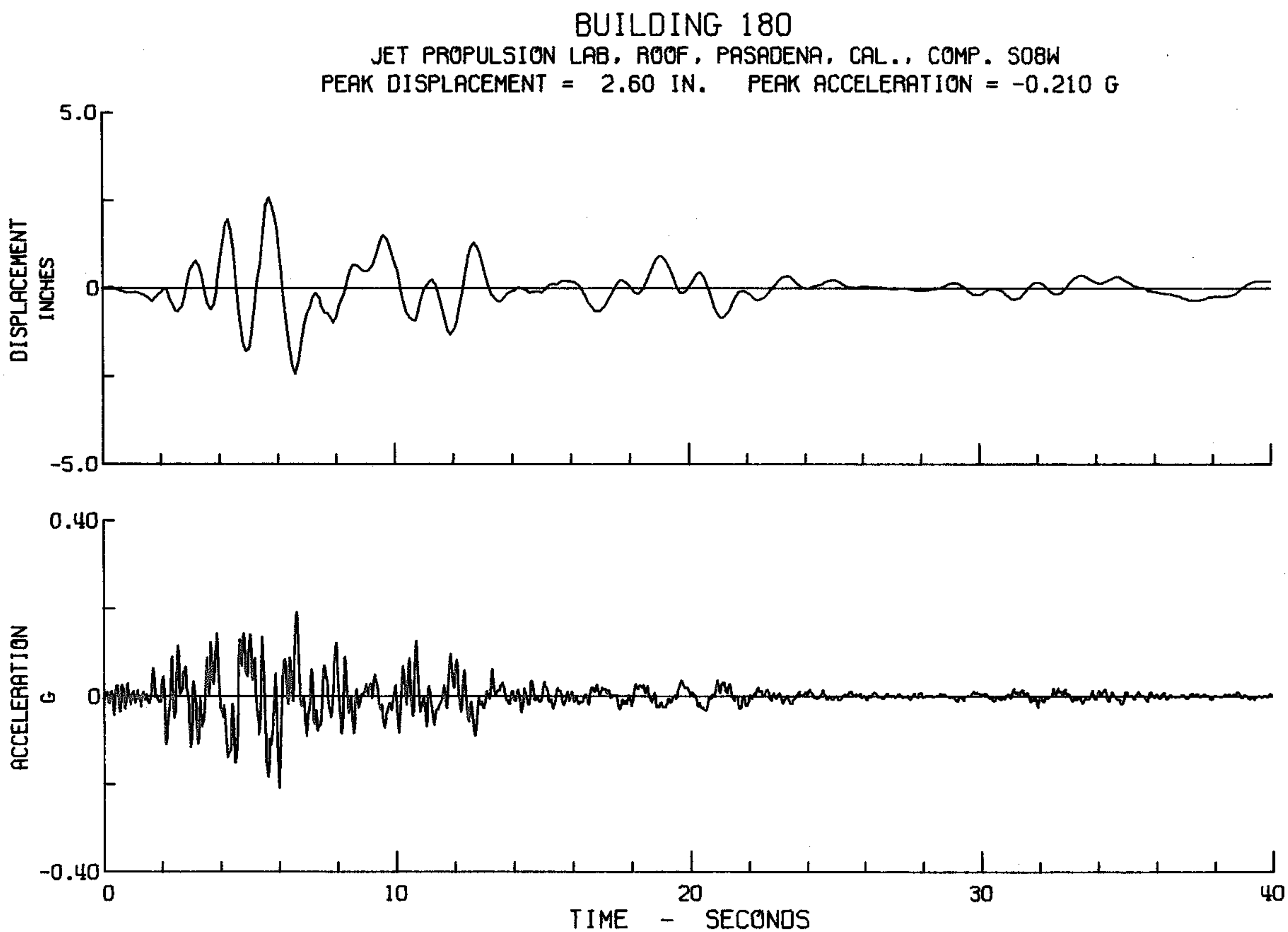

BUILDING 180
JET PROPULSION LAB, ROOF, PASADENA, CAL., COMP. S08W
PEAK DISPLACEMENT = 2.60 IN.   PEAK ACCELERATION = -0.210 G
DISPLACEMENT
INCHES
5.0
0
-5.0
ACCELERATION
G
0.40
0
-0.40
0
10
20
30
40
TIME - SECONDS

Figure 6.4

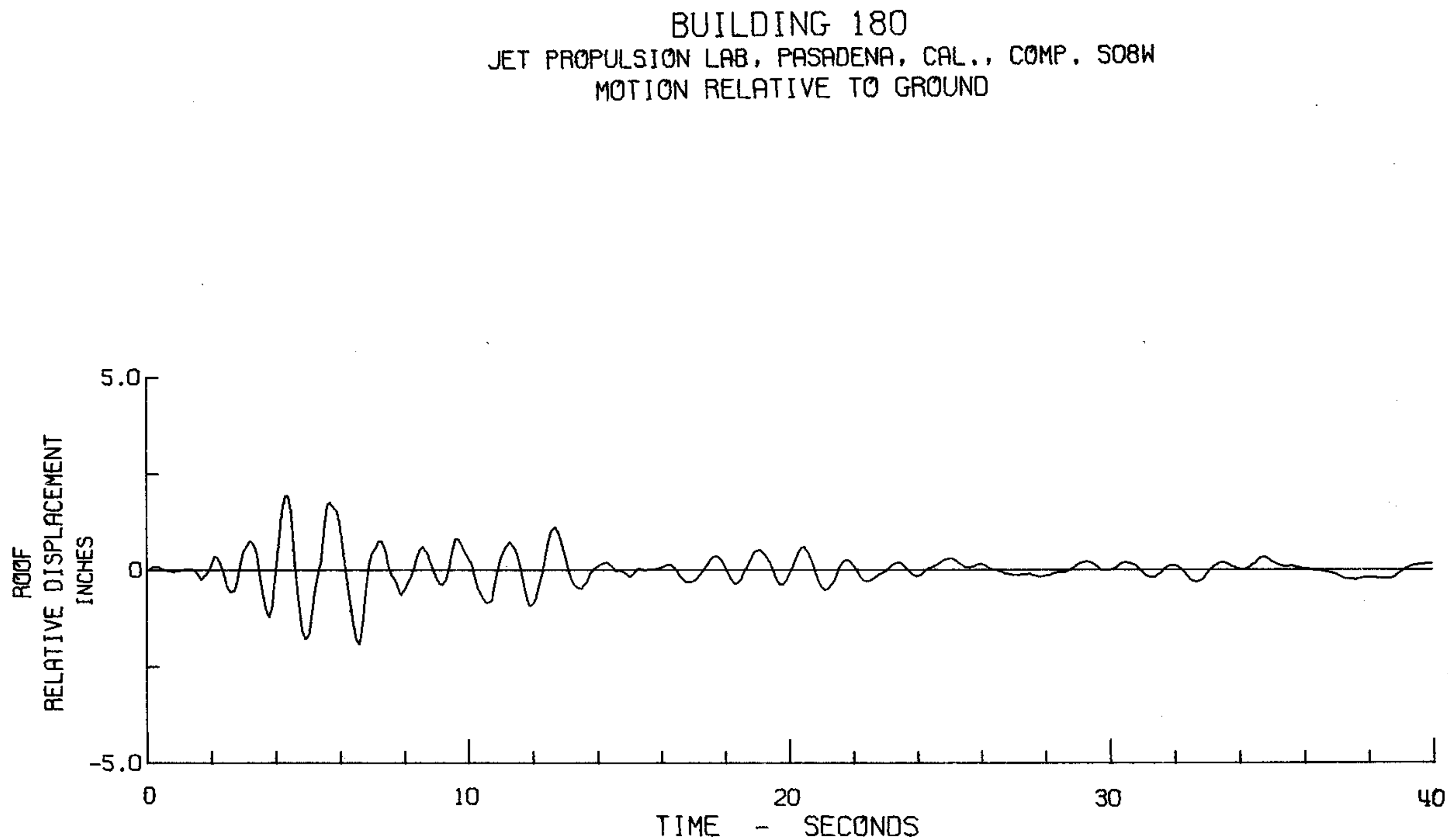

BUILDING 180
JET PROPULSION LAB, PASADENA, CAL., COMP. S08W
MOTION RELATIVE TO GROUND
ROOF
RELATIVE DISPLACEMENT
INCHES
5.0
0
-5.0
0
10
20
30
40
TIME - SECONDS

Figure 6.5

RESPONSE SPECTRUM

BUILDING 180

JET PROPULSION LAB, BASEMENT, PASADENA, CAL., COMP. S08W

DAMPING VALUES ARE 0, 2, 5, 10 AND 20 PERCENT OF CRITICAL

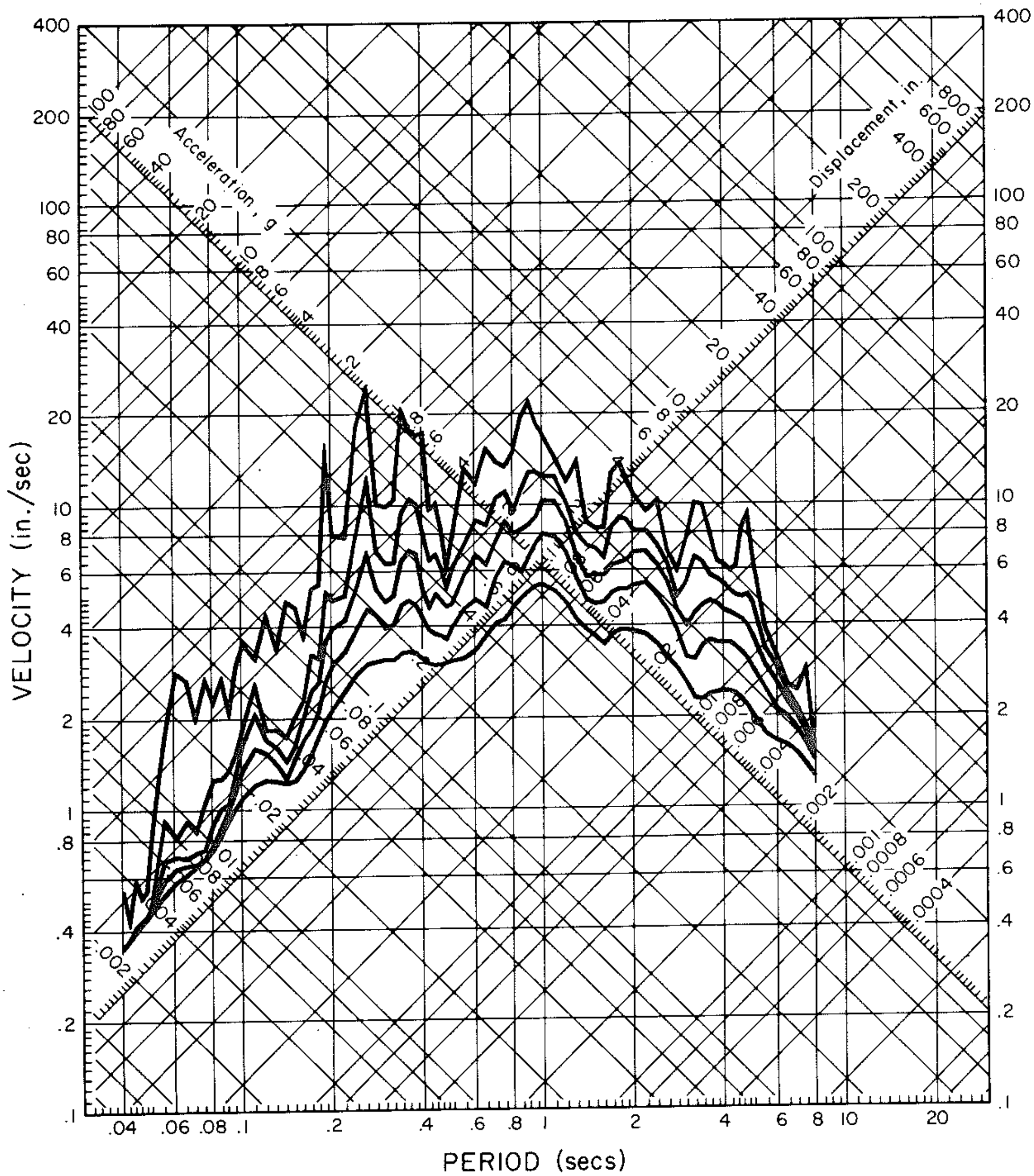


Figure 6.6

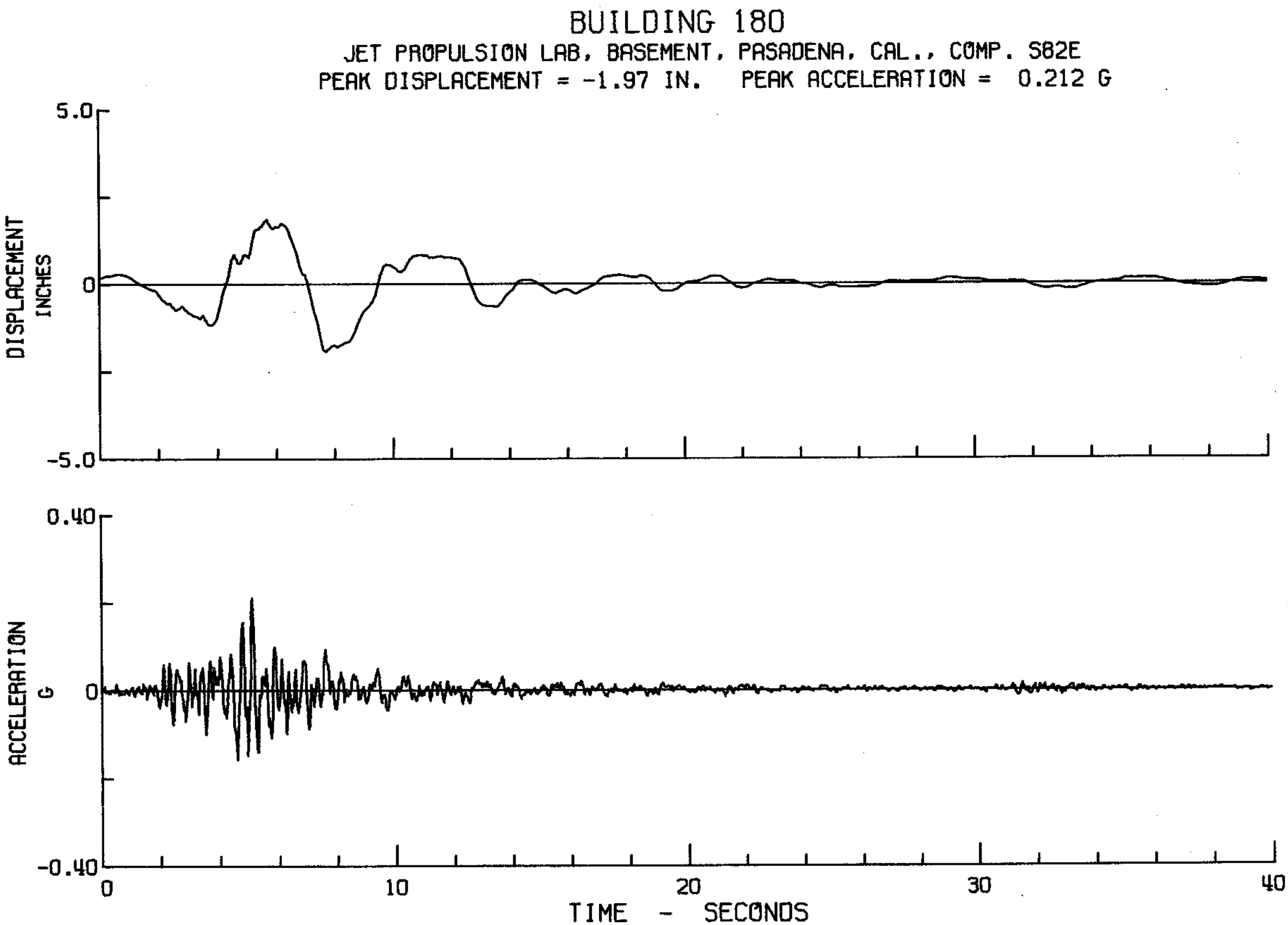

BUILDING 180
JET PROPULSION LAB, BASEMENT, PASADENA, CAL., COMP. S82E
PEAK DISPLACEMENT = -1.97 IN.
PEAK ACCELERATION = 0.212 G
5.0
0
-5.0
DISPLACEMENT
INCHES
0.40
0
-0.40
ACCELERATION
G
0
10
20
30
40
TIME - SECONDS

Figure 6.7

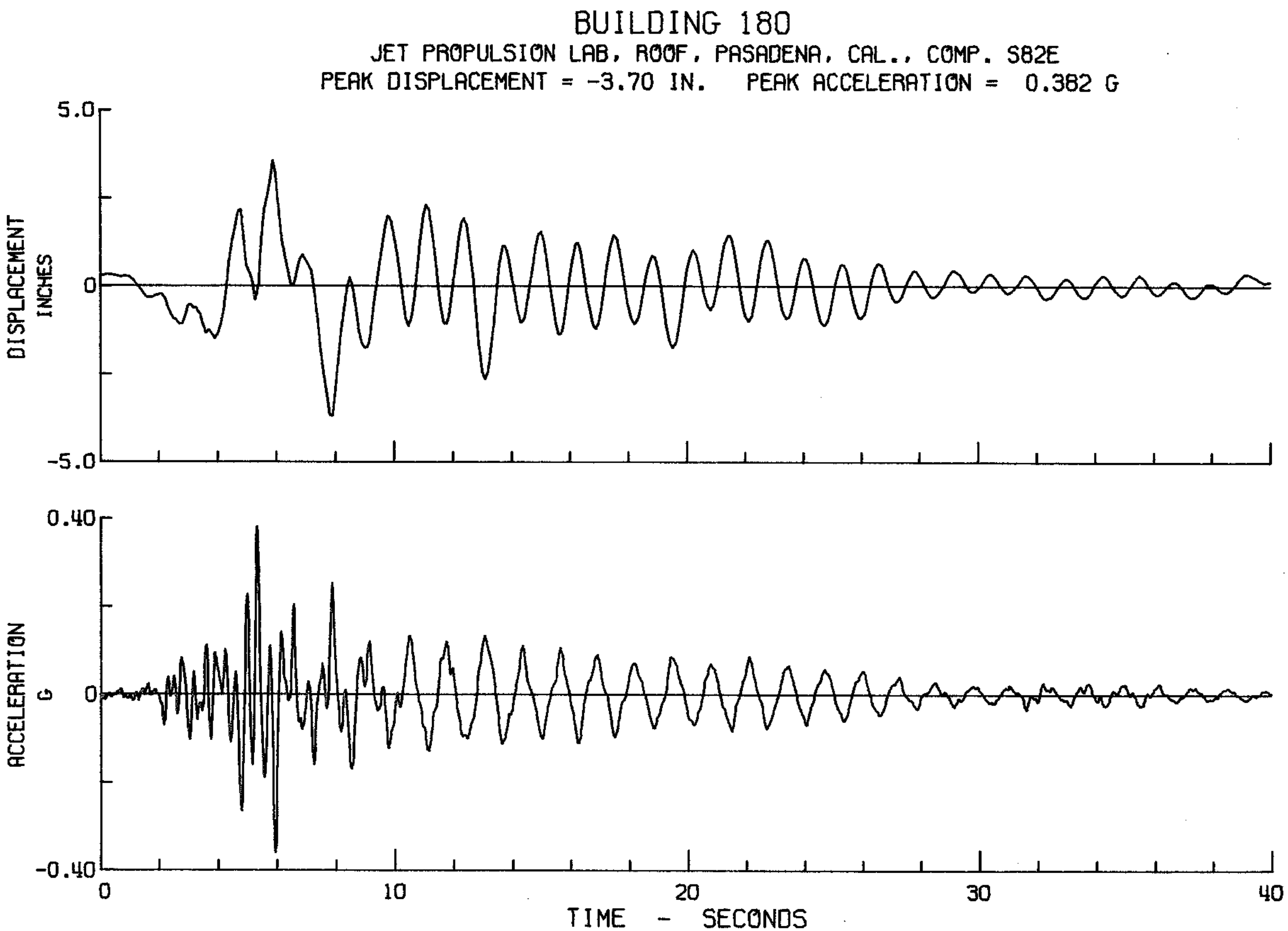

BUILDING 180
JET PROPULSION LAB, ROOF, PASADENA, CAL., COMP. S82E
PEAK DISPLACEMENT = -3.70 IN.   PEAK ACCELERATION = 0.382 G
DISPLACEMENT
INCHES
5.0
0
-5.0
ACCELERATION
G
0.40
0
-0.40
0
10
20
30
40
TIME - SECONDS

Figure 6.8

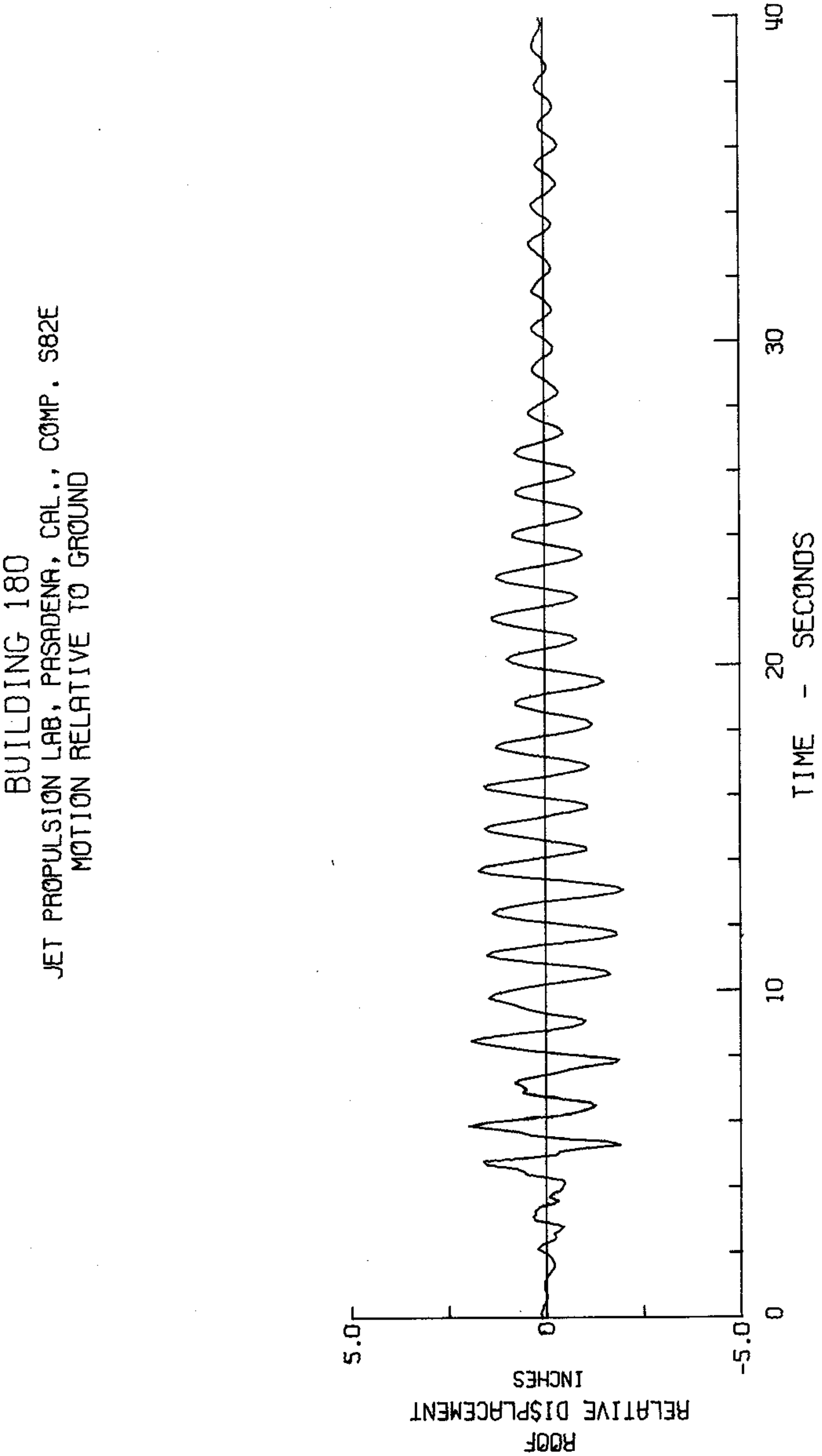

BUILDING 180
JET PROPULSION LAB, PASADENA, CAL., COMP. S82E
MOTION RELATIVE TO GROUND
TIME - SECONDS
0
10
20
30
40
5.0
0
-5.0
ROOF
RELATIVE DISPLACEMENT
INCHES

Figure 6.9

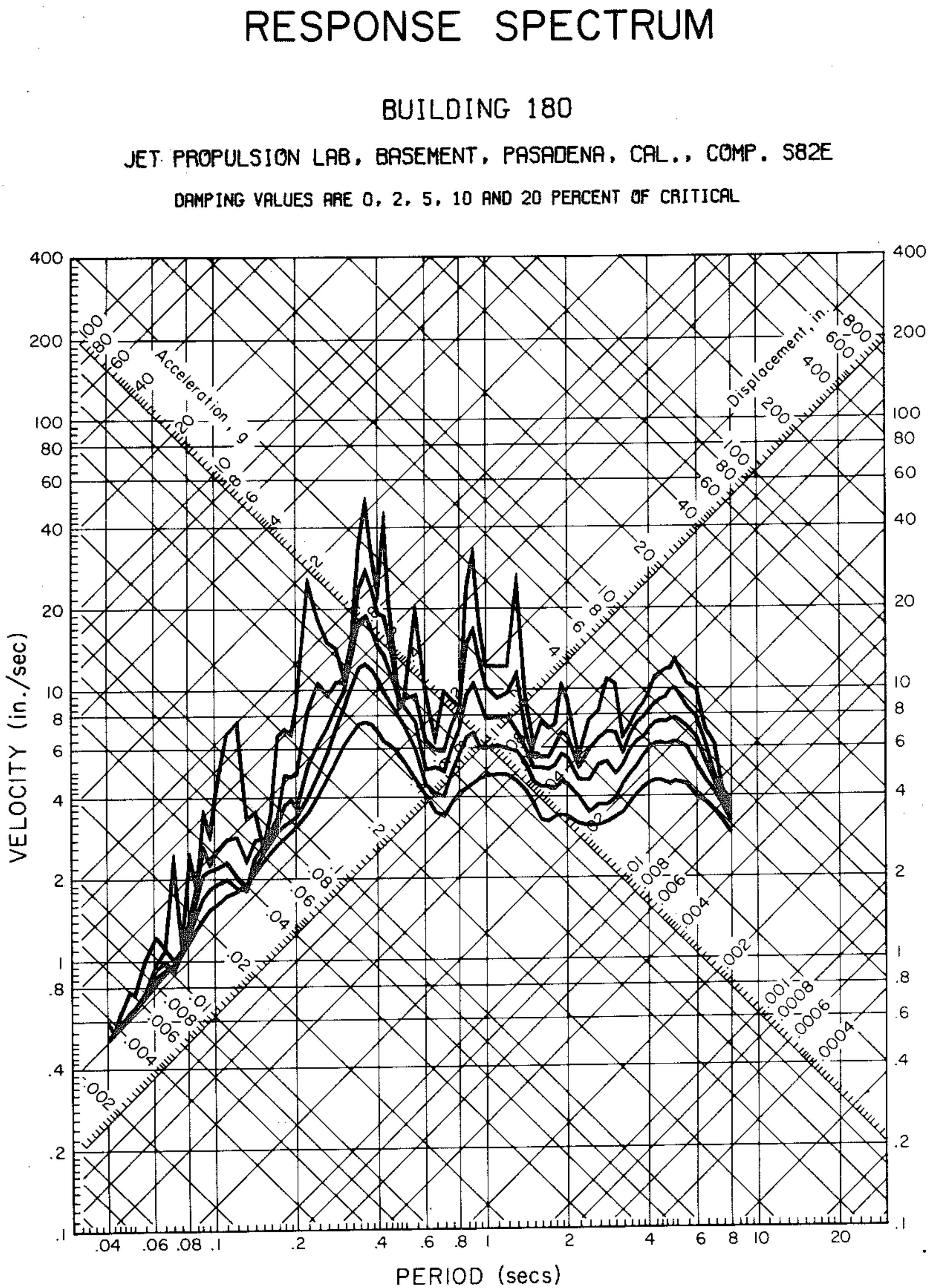

RESPONSE SPECTRUM
BUILDING 180
JET PROPULSION LAB, BASEMENT, PASADENA, CAL., COMP. S82E
DAMPING VALUES ARE 0, 2, 5, 10 AND 20 PERCENT OF CRITICAL
VELOCITY (in./sec)
PERIOD (secs)
Acceleration, g
Displacement, in.

Figure 6.10

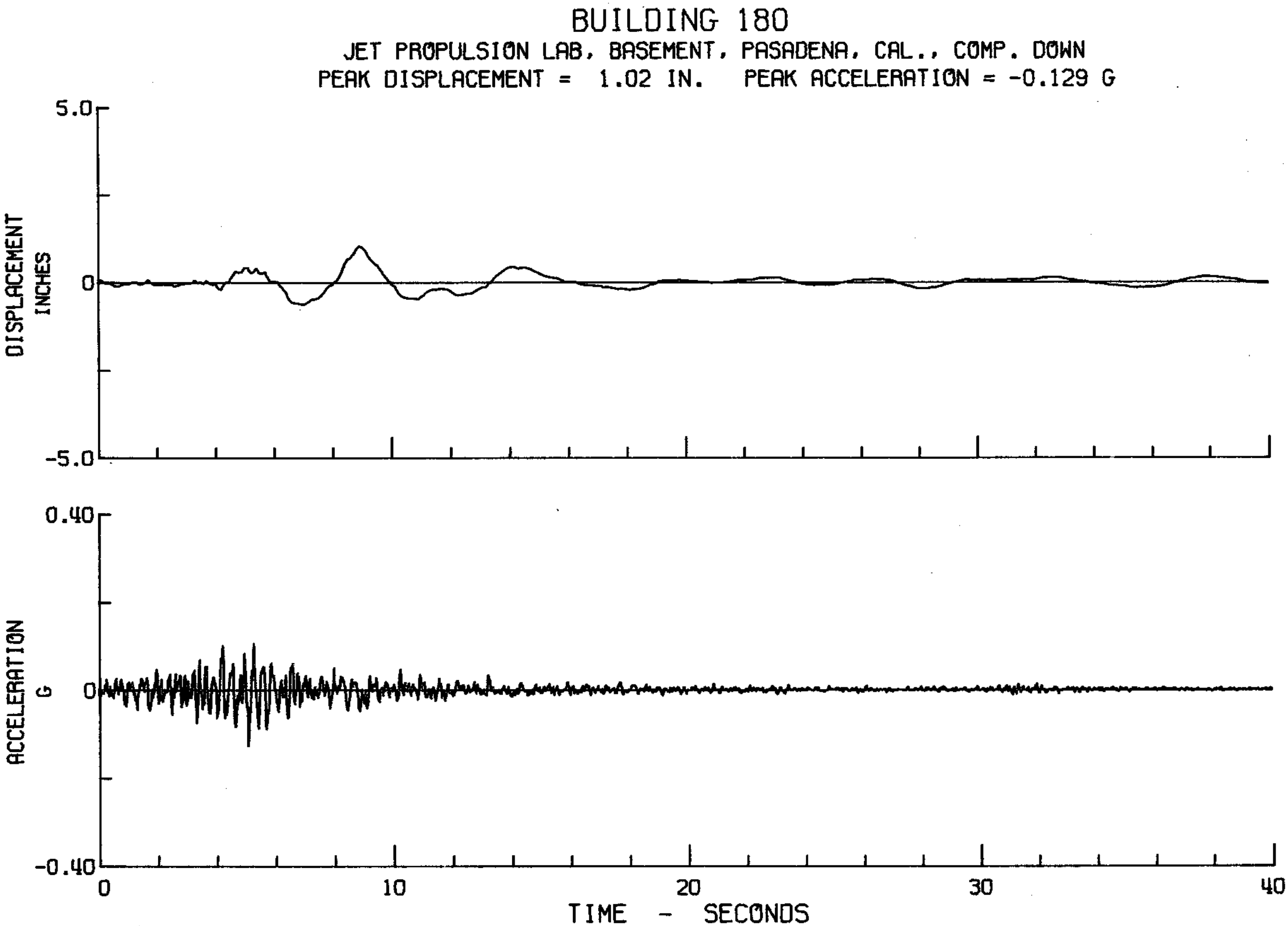

BUILDING 180
JET PROPULSION LAB, BASEMENT, PASADENA, CAL., COMP. DOWN
PEAK DISPLACEMENT = 1.02 IN. PEAK ACCELERATION = -0.129 G
5.0
0
-5.0
DISPLACEMENT
INCHES
0.40
0
-0.40
ACCELERATION
G
0
10
20
30
40
TIME - SECONDS

Figure 6.11

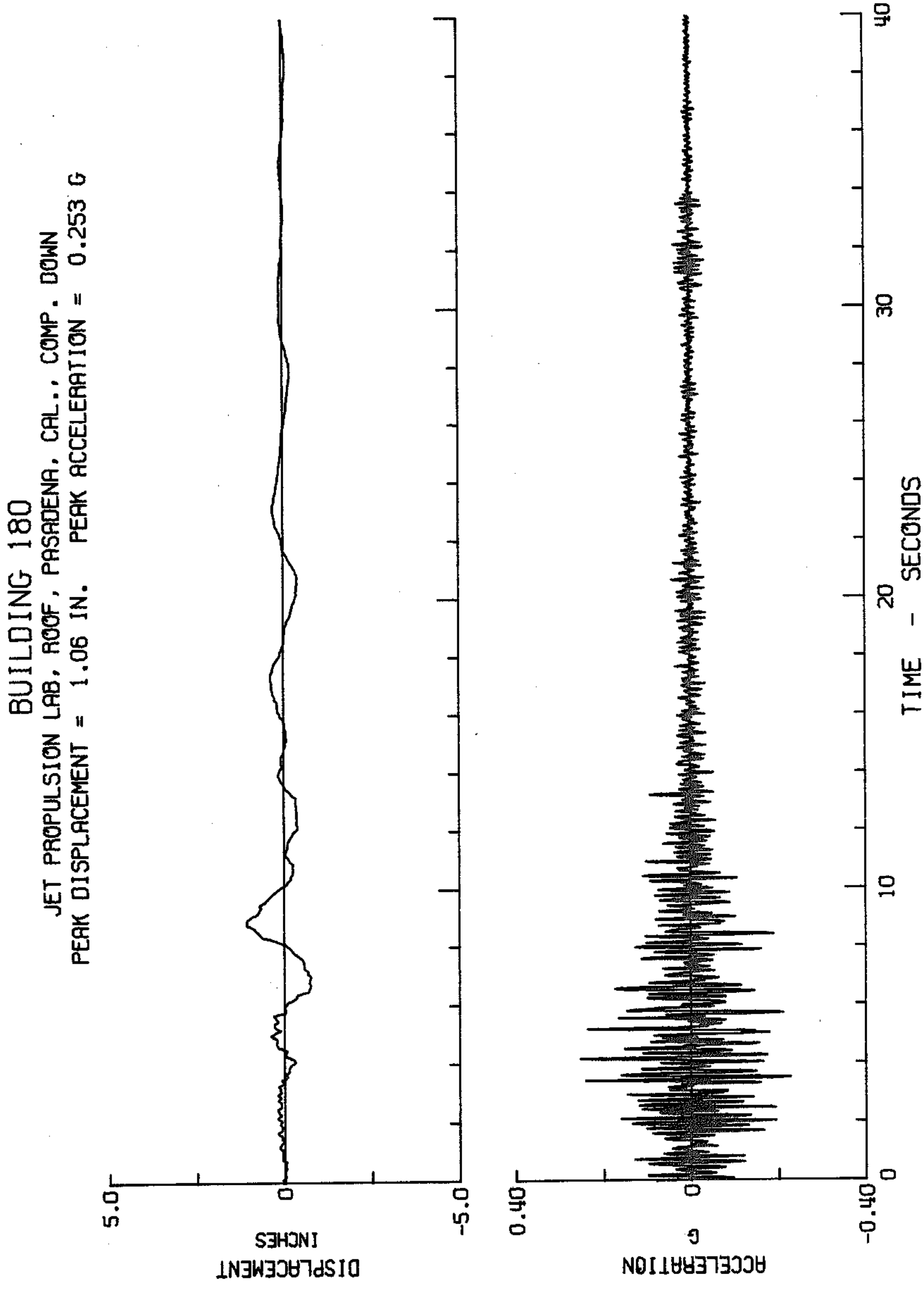

BUILDING 180
JET PROPULSION LAB, ROOF, PASADENA, CAL., COMP. DOWN
PEAK DISPLACEMENT = 1.06 IN.   PEAK ACCELERATION = 0.253 G
DISPLACEMENT
INCHES
5.0
0
-5.0
ACCELERATION
G
0.40
0
-0.40
0
10
20
30
40
TIME - SECONDS

Figure 6.12

RESPONSE SPECTRUM

BUILDING 180

JET PROPULSION LAB, BASEMENT, PASADENA, CAL., COMP. DOWN

DAMPING VALUES ARE 0, 2, 5, 10 AND 20 PERCENT OF CRITICAL

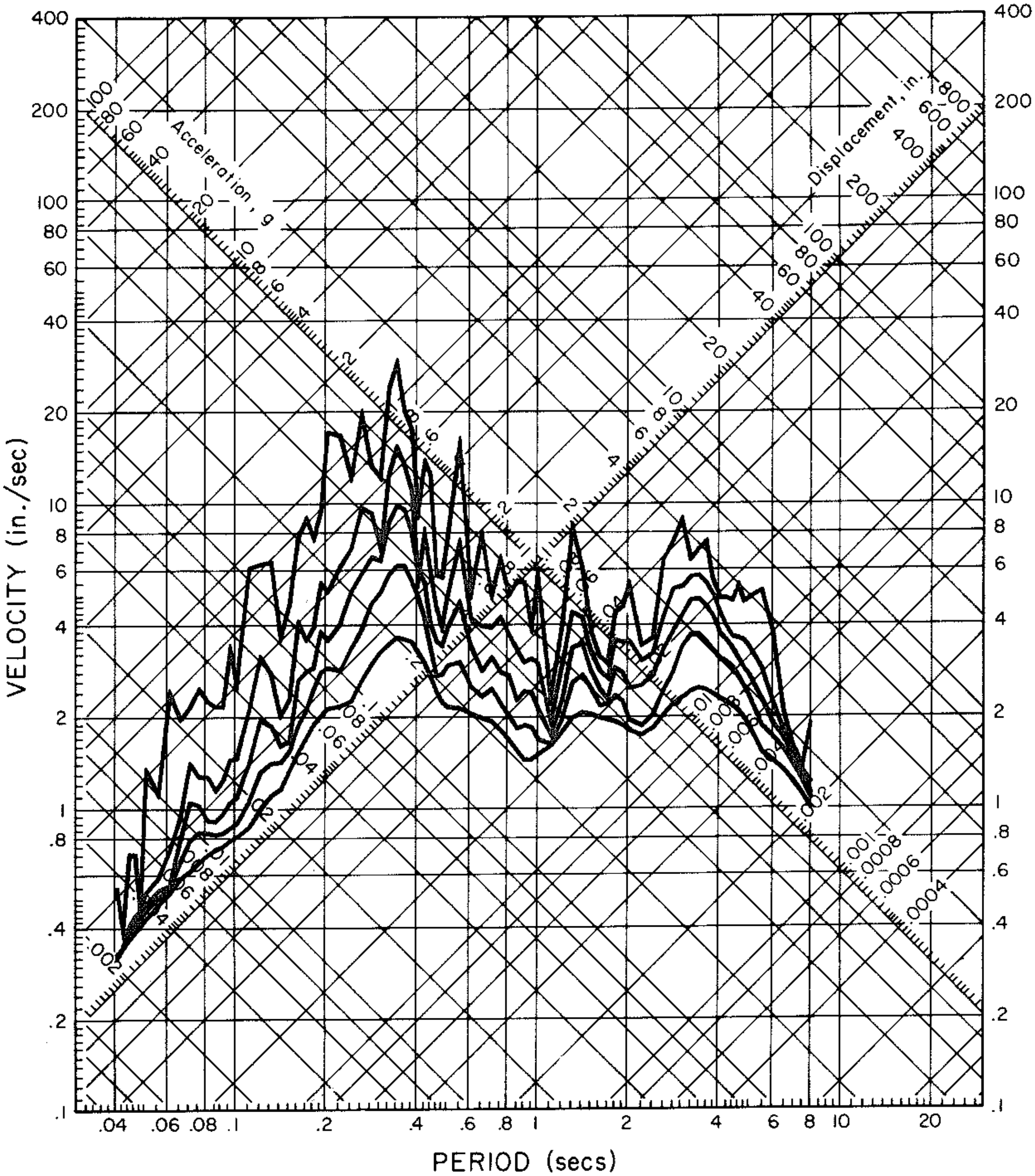


Figure 6.13